MISCELLANEOUS PAPER EL-79-6

US Army Corps of Engineers

# MILITARY HYDROLOGY

## REPORT 21

# REGULATION OF STREAMFLOW BY DAMS AND ASSOCIATED MODELING CAPABILITIES

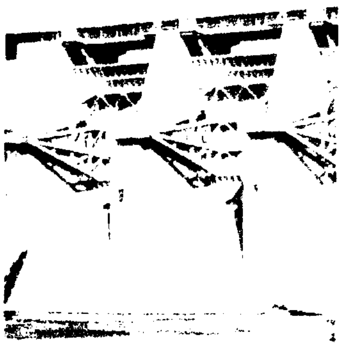
by

Ralph A. Wurbs

Environmental Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199





October 1992
Report 21 of a Series





Prepared for DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under DA Project No. 4A762719AT40
Task Area BO, Work Unit 052

# REPORT DOCUMENTATION PAGE

*Form Approved*
*OMB No. 0704-0188*



**1. AGENCY USE ONLY** *(Leave blank)*

**2. REPORT DATE**
October 1992

**3. REPORT TYPE AND DATES COVERED**
Report 21 of a series

**4. TITLE AND SUBTITLE**
Military Hydrology; Report 21, Regulation of Streamflow by Dams and Associated Modeling Capabilities

**5. FUNDING NUMBERS**
DA 4A762719 AT40
Task Area BO
Work Unit 052

**6. AUTHOR(S)**
Ralph A. Wurbs

**7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)**
U.S. Army Engineer Waterways Experiment Station
Environmental Laboratory
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

**8. PERFORMING ORGANIZATION REPORT NUMBER**
Miscellaneous Paper EL-79-6

**9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)**
U.S. Army Corps of Engineers
Washington, DC 20314-1000

**10. SPONSORING/MONITORING AGENCY REPORT NUMBER**

**11. SUPPLEMENTARY NOTES**
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

**12a. DISTRIBUTION/AVAILABILITY STATEMENT**
Approved for public release; distribution is unlimited.

**12b. DISTRIBUTION CODE**

**13. ABSTRACT** *(Maximum 200 words)*

This report provides a general overview of dams and reservoirs, how they are operated to regulate streamflow, and mathematical modeling capabilities for analyzing their operation. The report is based on a review of the literature, with a view toward developing information pertinent to military planning and operations. The treatment of dams and reservoirs includes: (a) a discussion of the worldwide inventory of dams, (b) types and configurations of dams and appurtenant structures, and (c) reservoir regulation procedures. State-of-the-art modeling capabilities are reviewed in the categories of stimulation, optimization, flood wave analysis, streamflow synthesis, water quality, and sediment transport. Development of rating curves is also covered.

**14. SUBJECT TERMS**
Dams
Military hydrology
Regulation
Streamflow

**15. NUMBER OF PAGES**
122

**16. PRICE CODE**

| 17. SECURITY CLASSIFICATION OF REPORT | 18. SECURITY CLASSIFICATION OF THIS PAGE | 19. SECURITY CLASSIFICATION OF ABSTRACT | 20. LIMITATION OF ABSTRACT |
|---|---|---|---|
| UNCLASSIFIED | UNCLASSIFIED | | |

# PREFACE

The work reported herein was conducted under Department of the Army Project No. 4A762719AT40, "Mobility and Weapons Effects Technology," Task Area BO, "AirLand Battlefield Environment," Mission Area, "Combat Support," Work Unit 052, "Induced Floods as Linear/Area Obstacles," under the auspices of the Battlefield Terrain Working Group of the AirLand Battlefield Environment Thrust. The study was sponsored by the Headquarters, U.S. Army Corps of Engineers (HQUSACE). MAJ Denton Brown was the HQUSACE Technical Monitor.

The study was conducted by the U.S. Army Engineer Waterways Experiment Station (WES) under the general supervision of Dr. John Harrison, Director of the Environmental Laboratory (EL), and Dr. V. E. LaGarde III, Chief of the Environmental Systems Division (ESD), EL, and under the direct supervision of Mr. M. P. Keown, Chief of the Environmental Constraints Group (ECG), ESD, EL, and Mr. J. C. Collins, ECG. Mr. M. R. Jourdan, ECG, Principal Investigator, Work Unit 052, provided technical assistance and review. This report was prepared by Dr. Ralph A. Wurbs, who is an Associate Professor at Texas A&M University working under an Intergovernmental Personnel Act agreement as a Research Engineer, ECG.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Leonard G. Hassell, EN.

This report should be cited as follows:

Wurbs, Ralph A. 1992. "Military Hydrology; Report 21, Regulation of Streamflow by Dams and Associated Modeling Capabilities," Miscellaneous Paper EL-79-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

CONTENTS


| | Page |
|---|---|
| PREFACE | 1 |
| PART I: INTRODUCTION | 3 |
| Background | 3 |
| Military Significance of Dams and Reservoirs | 4 |
| Purpose and Scope | 5 |
| PART II: DAMS OF THE WORLD | 7 |
| History of Dams | 7 |
| Inventory of Dams | 13 |
| PART III: DAMS AND APPURTENANT STRUCTURES | 22 |
| Dams Types and Configurations | 22 |
| Spillways | 26 |
| Outlet Works | 31 |
| Other Structures | 35 |
| PART IV: RESERVOIR OPERATION | 37 |
| Reservoir Pools | 37 |
| Multiple Purpose Reservoir Operation | 38 |
| Operation for Conservation Purposes | 40 |
| Flood Control Operations | 44 |
| PART V: SPILLWAY AND OUTLET WORKS RATING CURVES | 51 |
| References and Computer Programs | 51 |
| Basic Equations | 52 |
| Discharge Over a Spillway | 57 |
| Discharge Through an Outlet Works | 63 |
| PART VI: MODELING RESERVOIR OPERATIONS | 65 |
| Simulation of Reservoir Operations | 66 |
| Optimization Models | 78 |
| Comparison of Simulation and Optimization Models | 82 |
| Flood Wave Analysis | 84 |
| Watershed Modeling | 99 |
| Synthetic Streamflow Generation | 102 |
| Water Quality Modeling | 105 |
| Sediment Transport | 109 |
| PART VII: SUMMARY AND CONCLUSIONS | 112 |
| PART VIII: RECOMMENDATIONS | 113 |
| REFERENCES | 114 |

# MILITARY HYDROLOGY

# REGULATION OF STREAMFLOW BY DAMS AND ASSOCIATED MODELING CAPABILITIES

## PART I: INTRODUCTION

### Background

1. Under the Meteorological/Environmental Plan for Action, Phase II, approved for implementation on 26 January 1983, the US Army Corps of Engineers (USACE) has been tasked to implement a research, development, testing, and evaluation program that will: (a) provide the Army with environmental effects information needed to operate in a realistic battlefield environment, and (b) provide the Army with the capability for near-real time environmental effects assessment on military material and operations in combat. In response to this tasking, the Directorate for Research and Development, USACE, initiated the AirLand Battlefield Environment (ALBE) Thrust program. This initiative is developing the technologies to provide the field Army with the operational capability to perform and exploit battlefield effects assessments for tactical advantage.

2. Military hydrology, one facet of the ALBE Thrust, is a specialized field of study that deals with the effects of surface and subsurface water on planning and conducting military operations. In 1977, the Office, Chief of Engineers, approved the Military Hydrology Research Program. Management responsibility was subsequently assigned to the Environmental Laboratory, US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi.

3. The objective of military hydrology research is to develop an improved hydrologic capability for the Armed Forces with emphasis on applications in the tactical environment. To meet this overall objective, research is being conducted in four areas: (a) weather-hydrology interactions, (b) state of the ground, (c) streamflow, and (d) water supply. This report addresses streamflow.

## Military Significance of Dams and Reservoirs

4. Most major rivers throughout the world are regulated by systems of dams and reservoirs. Streamflow conditions are highly dependent upon man's operation of reservoirs as well as nature's provision of precipitation. Dams are necessary to control flooding and utilize the surface water resource for beneficial purposes such as agricultural, municipal and industrial water supply, hydroelectric power generation, and navigation. Although dams have been constructed for thousands of years, a tremendous growth in the number and size of dams has occurred during the past half-century.

5. Effective management of surface water resources is crucial to the economic vitality of regions and nations. Water supply, food production, electrical energy, transportation, and other functions served by reservoirs are important to military as well as civilian endeavors. These services are particularly important during wartime when efficiency and productivity must be maximized under adverse conditions.

6. Dams are potential targets for attacks, including terrorism during peacetime as well as military actions during war. Modern weapons provide the capability to inflict all relevant degrees of damage to a dam, ranging from the jamming of a spillway gate to complete destruction of the dam. Loss of the services provided by a dam can in many cases seriously diminish industrial productivity and overall support of a war effort. Downstream flooding caused by demolition of high dams on many rivers throughout the world can cause catastrophic damage and loss of life. The potential for damage from reservoir releases has increased dramatically since World War II with the increase in number and size of dams, development of floodplains below dams, and destructive potential of modern weapons. Water supply reservoirs are also subject to chemical contamination as a means of inflicting damage upon a population. Another military consideration is the potential for radiation contamination caused by a nuclear event in a large reservoir with subsequent inducement of radioactive "rain."

7. A potential deterrent to an attack on a strategically located dam is to partially empty the reservoir whenever a significant threat of attack is considered to exist. Although a controlled release would prevent or reduce downstream flooding if an attack did materialize, the precautionary drawdown can still interrupt the services provided by the reservoir. Drawdown plans can be developed to account for various inflow conditions, release

constraints, and impacts on reservoir services. The magnitude of the risk and consequences of an attack has to be balanced with the adverse consequences of emptying the reservoir as a defensive measure.

8. Reservoir gates can be operated or a dam breached to induce flooding during military operations. Under appropriate circumstances, reservoir releases can serve as an offensive weapon to damage and disrupt activities in the downstream floodplain. The obstacle effect of induced flooding can significantly strengthen defensive operations. Reservoirs can be effective in the rapid creation of barriers under expedient conditions. River-crossing operations in the combat zone may be delayed or prevented. The presence of a dam in a headwaters area under the control of the opposing force may necessitate the assembly and construction of river-crossing equipment capable of withstanding a major flood wave or series of flood waves, thereby acting as a deterrent to the operation. The obstacle effects of induced flooding include: (a) increasing velocities and stages to impede river-crossing operations, (b) destruction of bridges and other facilities, and (c) inundation of floodplain lands to adversely impact trafficability.

9. The reservoir itself may provide an obstacle to combat operations upstream of the dam. Situations can occur in which trade-offs exist between using a limited supply of water to maintain high water levels above the dam versus downstream induced flooding.

10. Combat operations can also be significantly impacted by streamflow conditions resulting from precipitation events. Reservoir operation is an important consideration in forecasting streamflow conditions to be expected from precipitation events. Discharges at a location on a river depend upon releases from upstream reservoirs and runoff from the uncontrolled watershed areas below the reservoirs. Backwater effects from downstream reservoirs can also be significant.

## Purpose and Scope

11. The objectives of this report are to provide general overviews of (a) dams and reservoirs, (b) the operations of dams and reservoirs to regulate streamflow, and (c) mathematical modeling capabilities available for analyzing the operations of dams and reservoirs. The report is based on a review of the literature. The treatment of dams and reservoirs includes: (a) a discussion of the worldwide inventory of dams (Part II), (b) types and configurations of

dams and appurtenant structures (Part III), and (c) reservoir regulation procedures (Part IV). Mathematical modeling capabilities for analyzing reservoir operations are outlined (Parts V and VI). Regulation of streamflow by dams can be analyzed from a variety of perspectives, all of which are pertinent to potential military applications. The intent here is to provide a comprehensive overview of the different types of modeling techniques. Basic fundamentals are summarized. References are provided for more in-depth study of each type of analysis.

## PART II: DAMS OF THE WORLD

### History of Dams

12. The history of dams closely follows the rise and decline of civilizations, especially in cultures highly dependent on irrigation. History does not record exactly when dams were first constructed. However, dams have served people for at least 5,000 years, beginning in the cradles of civilization of Babylonia, Egypt, India, Persia, and the Far East.

13. The oldest known dam is the Sadd el-Kafara, an Arabic name meaning "Dam of the Pagans," the ruins of which were discovered in 1885 at a location about 30 km south of Cairo, Egypt. This 11-m-high dam, which was probably built between 2950 and 2750 B.C., consisted of two rubble masonry walls running the full length of the dam with gravel and stone filling the space between the walls. The dam was constructed across a wadi apparently to provide a water supply for a stone quarry operation. After being in use for only a short time, the dam is believed to have been breached by a flood and was never rebuilt.

14. The Sadd el-Kafara is the only historically proven ancient Egyptian dam. King Menes, the founder of the first Egyptian dynasty, is reported to have dammed the Nile River in conjunction with the building of his capital city of Memphis about 2900 B.C. However, some historians consider this report to be only a legend.

15. In Babylonia and Assyria, irrigation was extensively developed in the Tigris and Euphrates Valleys as early as 2100 B.C. Marduk Dam was a notable ancient dam on the Tigris River north of Baghdad and south of Samaria dating to roughly 2000 B.C. It survived the Assyrian, Chaldean, Persian, Greek, Roman, and Sassarrian domination, but it breached and was left in ruin in the 13th century A.D.

16. One of the most impressive ancient water systems was developed in Judea by King Solomon between 1018 and 978 B.C. This system included a series of three reservoirs in a valley southwest of Jerusalem. Water was collected and stored in the reservoirs and then transported by aqueduct to the city.

17. The Romans have been called the greatest engineers of ancient times. Numerous dams were constructed all over the Roman empire.

18. The oldest major dam still in use today is the Almanza Dam in Spain. This 15-m-high arch dam, which was constructed in the 16th century, has been modified several times since its initial construction.

19. A comprehensive history of dams is provided by Smith (1971). Schnitter (1967) and Jansen (1980) present more concise coverages of the history of dams. Smith (1976) treats the history of water resources development in general.

20. Most major dams existing today were constructed after 1900. During the first half of the 20th century, dam development throughout the world experienced major changes. The institutional structure for water resources development and management was greatly expanded in many countries. Advances were made in all aspects of dam engineering and construction. Hydrologic, hydraulic, geotechnical, and structural analysis and design methods were developed. Construction techniques were greatly improved, and much larger dams were constructed than ever before.

21. Technological advancements in dam design and construction methods and the growth in number and size of dams initiated during the first half of the century have continued to the present. The rate of dam construction peaked during the 1950's and 1960's. The number of dams being constructed has decreased in recent years because of a number of factors including world-wide concern with environmental impacts, economics, and prior development of the best sites.

<u>Dam failures</u>

22. Ancient dam builders had only a very minimal understanding of the mechanics of materials or of flood flows, and by today's standards their methods were haphazard and their works often failed (Jansen 1980). Smith (1971) points out that the full impact of modern dam building technology was first felt around 1930. Since 1930, the failure rate has dropped sharply despite the fact that the number and size of dams has increased dramatically.

23. Jansen (1980) states that there have been perhaps 2,000 dam failures in the world since the 12th century A.D. Most of these were not major dams, however. Examples of dam failures with high associated death tolls include the San Ildefonso Dam failure in Bolivia in 1626, which may have resulted in the highest toll in human lives of any dam failure. The exact death toll is uncertain; however, estimates range as high as 4,000. The South Fork Dam above Johnstown, Pennsylvania, failed in 1889 with 2,209 lives lost. Machhu II Dam in India failed in 1979 with over 2,000 lives lost.

About 200 notable dam failures have occurred worldwide in the 20th century, with the loss of more than 8,000 people.

24. Biswas and Chatterjee (1971) concluded from a study of more than 300 dam failures throughout the world that about 35 percent were a direct result of floods in excess of the spillway capacity, and 25 percent were due to foundation problems such as seepage, piping, excessive pore pressures, inadequate cutoff, fault movement, settlement, or rockslides. The remaining 40 percent of the disasters were found to result from various problems including improper design or construction, inferior materials, wave action, acts of war, or general lack of proper operation and/or maintenance. Johnson and Illes (1976) estimate that about two percent of past dam failures can be attributed to intentional acts including acts of war.

25. In 1966, sabotage was suspected as a possible cause of the breaching of a dike impounding a sediment basin for a lead and zinc plant near Vratza in Bulgaria. The collapse of the earthfill embankment created a high flood wave through the towns of Zgorigrad and Vratza. Reports indicated that as many as 600 people perished, but the accepted record shows a death toll of 96.

Military actions involving dams

26. Early history. The first recorded use of a dam as a weapon of war occurred in 689 B.C. during the attack and destruction of Babylon by the Assyrian King Sennacherib. The Assyrians built a dam across the Euphrates River above the city. After a large lake had been impounded, the dam was breached with the resulting torrent of water destroying Babylon (Smith 1971). In 331 B.C., Alexander the Great led his forces into the Tigris River Valley. The records of his campaign indicate that dams on the river had to be partially removed to allow passage of his fleet (Jansen 1980). In the 13th century A.D., the French King Philip Augustus besieged the town of Gournay near Beauvais and hastened its surrender by breaching the dam which supplied water to the town's mills (Smith 1971). During the American Civil War, during the siege of Petersburg, the Confederate troops increased the strength of their defenses by damming a small creek to form a pond to serve as an obstacle (Dziuban 1947). The large gravity dams, the Burguillo and Ordunte, were attacked and damaged in 1937 during the Spanish Civil War.

27. World War II. Jansen (1980) provides fairly detailed accounts of a number of notable dam failures, including several military related dam breaches. Dziuban (1947) states that numerous instances of artificial

flooding occurred during World War II and cites several examples. Dziuban (1949a, 1949b, and 1950) further addresses various aspects of artificial flooding in military operations. The examples of World War II military actions involving dams cited below are based primarily on information provided by Jansen (1980) and Dziuban (1947).

28. Prior to 1939, the French Army installed the elaborate permanent fortifications of its Maginot Line along the Franco-German border from Switzerland to Luxembourg. The defenses included a system of dams to form a water obstacle. Gates were provided to operate the inundation system. The reservoir system, as well as most of the other fortifications, was taken intact and continued to be maintained by the Germans during the war.

29. Soviet troops withdrawing under German attack in September 1941 breached the Dnjeprostroj Dam in the southwest part of the Soviet Union. The intent was to delay the advancing German troops and allow Soviet forces time to reorganize. While the concrete gravity dam was still occupied by fleeing Soviet soldiers, about 30 trucks loaded with 2,700 kg of dynamite each were driven into a tunnel in the dam and exploded. The resulting breach was about 200 m wide with a maximum discharge of 35,000 $m^3$/sec. The Germans repaired the breach, but later, as the tide of war turned, sabotaged it themselves. The structure was damaged but not breached this time.

30. The Isoletta Dam in Italy is on the Liri River above its confluence with the Rapido River to form the Garigliano River. The British Army attempted to cross the Garigliano River in January 1944 while the Isoletta Dam was in Germany possession. The Germans released a flood wave from the dam which swept away British assault boats caught in midstream. The Germans continued releases from the reservoir which successfully prevented crossing of the river.

31. Several weeks later, the river still not crossed by Allied forces, the Americans attempted a crossing of the Rapido River which was not controlled by Isoletta Dam. The Germans dammed the river below the crossing site. The floodplain was converted into a quagmire. Many previously-laid mine fields were covered with water, rendering detection more difficult. Although the Allies were able to successfully complete the crossing, the artificial inundation of the valley seriously hindered their efforts.

32. Dziuban (1947) cites examples of artificial inundations at the Anzio beachhead in 1944. Artificial inundations were also of considerable value to the German defenders during the Allied landings and subsequent

operations at Normandy in 1944. However, these inundations did not involve major dams.

33. The operations against the Mohne, Eder, and Sorpe Dams in Germany probably represent the foremost example of the use by the Allies of dam destruction and induced flooding from both the strategic and offensive perspectives. The dams were critical components of a system for supplying water and hydroelectric power for the vital Ruhr industrial complex. Early in the war, the British War Cabinet, realizing the significance of the dams to German war production, made them targets for destruction. An extensive planning effort culminated in the British Air Force bombing the dams using special heavy rotating bombs, called roll bombs, in a low-level surprise attack in May 1943. The Mohne and Eder Dams, which were concrete gravity structures, were breached. The earthen Sorpe Dam took two direct hits but did not breach. The flood wave caused by breaching the Mohne and Eder Dams resulted in widespread devastation and the loss of 1,200 lives. Both dams were repaired after the attack. The Sorpe Dam was bombed several more times during the war, but although significantly damaged, remained in service.

34. The Etang de Lindres Dam, a large earthen embankment on the Seille River in France, was breached by German fighter bombers in October 1944. However, the Allies had been aware of the possibility of breaching the dam and had made preparations, involving the removal of four bridges, including two Baileys, and reinforcing and securing of remaining bridges. These preparations were considered successful.

35. In the Fall of 1944, the Americans were preparing plans for crossing the Rhine River into Germany. The detailed studies and analyses included a physical model study of operation of the system of reservoirs on the Rhine (Dziuban 1946). The Germans had earlier developed detailed plans for the use of induced flood waves to create barriers, and the Allies were well aware of this potential use of the dams.

36. Seven dams located on the international boundary between Germany and Switzerland were normally operated in accordance with agreements between the two countries for hydroelectric power and navigation. The location of the international boundary divided the control of the outlet gates between the Germans and Swiss. The point on each dam marking the international boundary was strongly closed off by fences, heavy barbed wire, and similar obstacles. In order to prevent induced flooding in case the Germans seized control of the dams, the Swiss had artillery sited to fire on and damage the gates so that

they would not operate. Two other dams were under complete control of the Germans. Of the nine dams, eight were equipped with sluice gates capable of releasing catastrophic floods. The ninth dam would have to have been breached to induce severe flooding. By repeatedly opening and closing the gates at the dams, the Rhine River could be maintained as an effective barrier. The modeling experiments conducted by the Americans analyzed the induced flooding that the Germans could achieve if they seized control of all the dams.

37. The characteristics of flood waves induced by repeatedly opening and closing the gates were modeled mathematically. However, the limitations of the mathematical methods available resulted in very approximate results. Consequently, a physical model of the reservoir-stream system was constructed at the Neyret-Beylier et Picard-Pictet Laboratory. Numerous test runs were made with various combinations of natural discharge and gate manipulation strategies. In November 1944, British bombers destroyed or damaged several gates at one of the dams. Later as the French Army seized control of a portion of the west bank of the Rhine, the Germans demolished more gates. The model experiments were quickly adjusted to provide data under these new conditions.

38. The planning and preparation for the Rhine crossing also included establishment of the Rhine River Flood Prediction Service (Dziuban 1945). This service maintained streamflow and rainfall gaging stations and developed short-range and long-range predictions of river stages. The engineers of the Allied Armies were provided constant information regarding river conditions which supported decisions regarding optimal dates and times to undertake assaults and measures to protect floating bridges and other crossing equipment.

39. Korean War. The 15-m high Taksan Dam, an earthen structure in North Korea, was attacked in May 1953 by 20 United Nations fighter bombers, probably using 230-kg general purpose bombs (Jansen 1980). Erosion collapsed a large section of the dam. The 2.4-km-long reservoir was completely emptied, flooding the valley for 43 km downstream. The flood destroyed or damaged 10 rail and highway bridges, flooded an airfield, and washed out or silted up miles of irrigation canals.

40. Three days after the bombing of the Taksan Dam, dive-bombing attacks were made against Chasan Dam which was similar in size to Taksan Dam. Five hits occurred in a concentrated area to create an initial breach, which was widened by erosion. The flood destroyed or severely damaged 4 km of main

railway line, a rail switching yard, 4.8 km of highway, 70 buildings, two major bridges, and several kilometers of irrigation canals. The main supply routes to the south were cut for two weeks, and extensive and irreparable damage was done to the rice crop, which formed the basis of the food supply for the nation.

41. The 81-m-high Hwacheon Dam, a concrete gravity structure on the North Han River just above the 38th parallel, was included in the battle tactics of both sides during the Korean War. The North Koreans used the spillway gates of the dam to release a flood wave downstream, resulting in the destruction of one floating bridge and the removal of another. These two bridges were the only available crossings on a important highway leading north. After this experience with an artificial flood, the US Navy sent torpedo bombers to destroy some of the Hwacheon Dam spillway crest gates. Several gates were destroyed, preventing the filling of the reservoir and, thus, its use for flooding purposes.

42. Present significance. The previous examples illustrate a variety of ways that dams have impacted military operations in the past. The military significance of dams has greatly increased since World War II because of a number of factors. A tremendous growth in the number and size of dams has occurred since that time. Streamflow in the major rivers throughout the world is now controlled to a much greater extent by dams. Increasing population results in a greater dependence on the water, food, and energy resources provided by dams. The destructive potential of flood waves which would result from breaching, or in many cases gate releases, is catastrophic. The development of more potent weapons, including nuclear bombs, has increased the potential for destroying dams.

## Inventory of Dams

43. Most publications describing existing dams are directed to a specific nation, region, or river basin. The primary source of published data covering the worldwide inventory of dams is the International Commission on Large Dams (ICOLD) which was founded in France in 1928. Its headquarters are located in Paris. The primary objective of the ICOLD is to encourage improvements in the design, construction, maintenance, a. operation of large dams by (a) organizing technical studies and research, (b) interchanging of

information amongst various member countries, and (c) holding executive meetings annually and an international congress each third year.

44. The ICOLD developed and maintains the "World Register of Dams," a listing of large dams with associated information on type, dimensions, and ownership. Major editions of the register were published in 1964, 1973, and 1984. Several updatings were published between major editions. The register is a compilation of data developed by national committees in the participating countries. For purposes of inclusion in the register, the ICOLD has defined a large dam as either:

a. A dam above 15 m in height, measured from the lowest portion of the general foundation areas to the crest, or

b. A dam between 10 and 15 m in height provided it meets with at least one of the following conditions:

   (1) The length of the crest of the dam is greater than 500 m.

   (2) The capacity of the reservoir formed by the dam is greater than 1 million $m^3$.

   (3) The maximum flood discharge dealt with by the dam is greater than 2,000 $m^3$/sec.

   (4) The foundation problems associated with the dam were especially difficult.

   (5) The dam is of unusual design.

45. Mermel (1979) provides tables of data for 160 of the largest dams in the world. These tables and others found in various publications are based primarily on data compiled by the ICOLD.

46. The discussion below is based on the World Register of Dams (ICOLD 1984). Large dams meeting the above criteria are included in the register. The total number of small dames, i.e. those not meeting the above criteria for classification as large, is much greater than the number of large dams. Thus, the majority of the dams in the world are not included in the statistics quoted below.

Occurrence of large dams

47. The number of large dams in the world increased from 5,196 in the year 1950 to over 35,000 in 1982. Data constraints caused several hundred dams existing in 1982 to be omitted from the register, resulting in a total of 34,798 being reflected in the statistics quoted below. The 25 highest dams in the world are cited in Table 1. The 25 largest projects in terms of reservoir storage capacity are listed in Table 2.

Table 1

Highest Dams in the World

| No. | Height Above Lowest Foundation m | Type* | Name | Country | Year** |
|---|---|---|---|---|---|
| | 335 | TE/ER | Rogun | USSR | C |
| 1 | 300 | TE | Nurek | USSR | 1980 |
| 2 | 285 | PG | Grande Dixence | Switzerland | 1961 |
| 3 | 272 | VA | Inguri | USSR | 1980 |
| 4 | 262 | VA | Vajont | Italy | 1961 |
| 5 | 261 | ER | Manuel Moreno Torres (Chicoasen) | Mexico | 1980 |
| | 261 | ER/TE | Tehri | India | C |
| | 253 | ER/TE | Kishau | India | C |
| | 245 | VA/PG | Sayano-Shushensk | USSR | C |
| | 243 | ER | Guavio | Colombia | C |
| 6 | 242 | TE | Mica | Canada | 1972 |
| 7 | 237 | ER | Chivor | Colombia | 1975 |
| 8 | 237 | VA | Mauvoisin | Switzerland | 1957 |
| | 234 | VA | El Cajon | Honduras | C(1984) |
| 9 | 233 | VA | Chirkey | USSR | 1978 |
| 10 | 230 | TE | Oroville | USA | 1968 |
| 11 | 226 | PG | Bhakra | India | 1963 |
| 12 | 221 | VA/PG | Hoover | USA | 1936 |
| 13 | 220 | VA | Contra | Switzerland | 1965 |
| 14 | 220 | VA | Mratinje | Yugoslavia | 1976 |
| 15 | 219 | PG | Dworshak | USA | 1973 |
| 16 | 216 | VA | Glen Canyon | USA | 1966 |
| 17 | 215 | PG | Toktogul | USSR | 1978 |
| 18 | 214 | MV | Daniel Johnson | Canada | 1968 |
| 19 | 213 | VA | Dez | Iran | 1962 |
| | 210 | ER | San Roque | Philippines | C |
| 20 | 208 | VA | Luzzone | Switzerland | 1963 |
| 21 | 207 | ER/PG | Keban | Turkey | 1974 |
| 22 | 202 | VA | Almendra | Spain | 1970 |
| | 201 | VA | Khudoni | USSR | C |
| 23 | 200 | VA | Karoun | Iran | 1975 |
| 24 | 200 | VA | Kolnbrein | Austria | 1977 |
| 25 | 196 | PG/ER/TE | Itaipu | Brazil | 1982 |
| | 195 | ER | Altinkaya | Turkey | C |
| 26 | 194 | VA | New Bullard's Bar | USA | 1979 |
| | 192 | PG | Lakhwar | India | C |
| 27 | 191 | ER | New Melones | USA | 1979 |
| 28 | 186 | VA | Kurobe | Japan | 1964 |

(Continued)

SOURCE: 1984 World Register of Dams

* Dam type is defined as follows: TE - earth; ER - rockfill; PG - gravity; CB - buttress; VA - arch; MV - multiarch.

** C indicates the dam is under construction.

(Sheet 1 of 3)

Table 1 (Continued)

| No. | Height Above Lowest Foundation m | Type | Name | Country | Year |
|---|---|---|---|---|---|
| 29 | 186 | TE | Swift | USA | 1958 |
| | 186 | VA | Zillergrundl | Austria | C |
| 30 | 185 | VA | Mossyrock | USA | 1968 |
| | 185 | VA | Oymapinar | Turkey | C |
| | 184 | ER | Ataturk | Turkey | C |
| 31 | 183 | PG | Shasta | USA | 1945 |
| 32 | 183 | TE | W A C Bennett | Canada | 1967 |
| 33 | 180 | VA | Amir Kabir | Iran | 1964 |
| 34 | 180 | ER | Dartmouth | Australia | 1979 |
| 35 | 180 | VA | Emmosson | Switzerland | 1974 |
| | 180 | VA | Tehchi | Taiwan | 1974 |
| 36 | 180 | VA | Tignes | France | 1952 |
| 37 | 176 | ER | Takase | Japan | 1978 |
| 38 | 175 | ER | Ayvacik | Turkey | 1981 |
| 39 | 174 | PG | Alpe Gera | Italy | 1964 |
| 40 | 173 | TE | Don Pedro | USA | 1971 |
| | 173 | VA | Karakaya | Turkey | C |
| 41 | 172 | VA | Hungry Horse | USA | 1953 |
| | 172 | PG | Longyangxia | China | C |
| | 171 | VA | Cabora Bassa | Mozambique | 1974 |
| 42 | 169 | VA | Idukki | India | 1974 |
| 43 | 168 | ER | Charvak | USSR | 1977 |
| | 168 | ER | Gura Apelor | Romania | C |
| 44 | 168 | ER | La Grande 2 | Canada | 1978 |
| 45 | 168 | PG | Grand Coulee | USA | 1942 |
| 46 | 167 | ER | Fierze | Albania | 1978 |
| | 167 | VA/PG | Daniel Palacios | Ecuador | C |
| 47 | 166 | VA | Vidraru | Romania | 1965 |
| 48 | 165 | TE | Kremasta | Greece | 1965 |
| 49 | 165 | VA | Ross | USA | 1949 |
| 50 | 165 | PG | Wujiangdu | China | 1981 |
| | 164 | ER | Thomson | Australia | C |
| 51 | 164 | TE | Trinity | USA | 1962 |
| | 162 | PG/ER | Guri | Venezuela | C |
| 52 | 162 | ER | Talbingo | Australia | 1971 |
| 53 | 160 | ER | Foz de Areia | Brazil | 1980 |
| | 160 | TE/ER | Grand-Maison | France | C |
| | 160 | ER | Salvajina | Colombia | C |
| | 160 | ER/TE | Thein Dam Ranjit | India | C |
| 54 | 160 | VA | Yellowtail | USA | 1966 |
| | 158 | ER | Canales | Spain | C |
| | 158 | TE | Yacambu | Venezuela | C |
| 55 | 158 | ER | Cougar | USA | 1964 |
| 56 | 158 | ER | Emborcacao | Brazil | 1982 |

(Continued)

(Sheet 2 of 3)

Table 1 (Concluded)

| No. | Height Above Lowest Foundation m | Type | Name | Country | Year |
|---|---|---|---|---|---|
| 57 | 158 | VA | Gokcekaya | Turkey | 1972 |
| | 158 | ER | Naramata | Japan | C |
| | 157 | VA | Dongjiang | China | C |
| 58 | 157 | PG | Okutadami | Japan | 1961 |
| 59 | 157 | VA | Speccheri | Italy | 1957 |
| 60 | 156 | PG | Sakuma | Japan | 1956 |
| 61 | 156 | VA-TE | Zeuzier | Switzerland | 1957 |
| 62 | 155 | ER | Goescheneralp | Switzerland | 1960 |
| 63 | 155 | VA | Monteynard | France | 1962 |
| 64 | 155 | VA | Nagawado | Japan | 1969 |
| 65 | 155 | VA | Place Moulin | Italy | 1965 |
| | 155 | PG | Sadar Sarovar | India | C |
| 66 | 154 | VA/PG | Bhumibol | Thailand | 1964 |
| 67 | 154 | ER | Tedorigawa | Japan | 1979 |
| 68 | 153 | VA | Curnera | Switzerland | 1967 |
| 69 | 153 | VA | Flaming Gorge | USA | 1964 |
| 70 | 153 | ER | Gepatsch | Austria | 1965 |
| | 153 | PG/ER | Revelstoke | Canada | C |
| 71 | 153 | VA | Santa Giustina | Italy | 1950 |
| | 151 | PG | Dorna | Spain | C |
| | 151 | ER | Menzelet | Turkey | C |
| 72 | 151 | VA | Zervreila | Switzerland | 1957 |
| | 150 | PG | Baishan | China | C |
| 73 | 150 | VA | Canelles | Spain | 1960 |
| 74 | 150 | ER | Finstertal | Austria | 1980 |
| | 150 | ER | Kenyir | Malaysia | C |
| 75 | 150 | VA/CB | Roselend | France | 1961 |
| 76 | 150 | TE | Big Horn | Canada | 1972 |

| | | In Operation | Under Completion |
|---|---|---|---|
| Dams with h > 150 m | - ICOCD member countries | 76† | 32 |
| | - non-member countries | 2†† | 1‡ |

† 77 with Itaipu for Brazil and Paraguay.
†† Taiwan-Mozambique.
‡ Honduras.

(Sheet 3 of 3)

Table 2

World's Largest Reservoirs in Terms of Capacity

| No. | Capacity $10^6$ $m^3$ | Name | Country | Year |
|---|---|---|---|---|
| 1 | 204,800 | Owen Falls* | Uganda | 1954 |
| 2 | 169,270 | Bratsk | USSR | 1964 |
| 3 | 168,900 | High Aswan | Egypt | 1970 |
| 4 | 160,368 | Kariba | Zimbabwe/Zambia | 1959 |
| 5 | 147,960 | Akosombo | Ghana | 1965 |
| 6 | 141,851 | Daniel Johnson | Canada | 1968 |
| | 135,000 | Guri | Venezuela | C** |
| 7 | 73,300 | Krasnoyarsk | USSR | 1967 |
| 8 | 70,309 | W A C Bennett | Canada | 1967 |
| 9 | 68,400 | Zeya | USSR | 1978 |
| | 63,000 | Cabora Bassa | Mozambique | 1974 |
| 10 | 61,715 | La Grande 2 | Canada | 1978 |
| 11 | 60,020 | La Grande 3 | Canada | 1981 |
| 12 | 59,300 | Ust-Ilim | USSR | 1977 |
| 13 | 58,000 | Kuibyshev | USSR | 1955 |
| 14 | 53,790 | Caniapiscau Barrage KA 3 | Canada | 1980 |
| | 50,700 | Upper Wainganga | India | C** |
| 15 | 49,800 | Bukhtarma | USSR | 1960 |
| | 48,000 | Ataturk | Turkey | C** |
| 16 | 46,000 | Irkutsk | USSR | 1956 |
| | 43,000 | Tucurui | Brazil | C** |
| 17 | 35,900 | Vilyui | USSR | 1967 |
| 18 | 35,400 | Sanmenxia | China | 1960 |
| 19 | 34,852 | Hoover | USA | 1936 |
| 20 | 34,100 | Sobradinho | Brazil | 1979 |
| 21 | 33,304 | Glen Canyon | USA | 1966 |
| 22 | 32,203 | Skins Lake No 1 | Canada | 1953 |
| 23 | 31,790 | Jenpeg | Canada | 1975 |
| 24 | 31,500 | Volgograd | USSR | 1958 |
| | 31,300 | Sayano-Shushensk | USSR | C** |
| 25 | 30,600 | Keban | Turkey | 1974 |
| 26 | 29,959 | Iroquois | Canada | 1958 |
| 27 | 29,000 | Itaipu | Brazil | 1982 |
| 28 | 28,973 | Churchill Falls (GR-1) | Canada | 1971 |
| 29 | 28,370 | Missi Falls Control | Canada | 1976 |
| 30 | 28,100 | Kapchagay | USSR | 1970 |
| 31 | 29,000 | Loma de la Lata | Argentina | 1977 |
| 32 | 27,920 | Garrison | USA | 1953 |
| 33 | 27,675 | Kossou | Ivory Coast | 1972 |

(Continued)

SOURCE: 1984 World Register of Dams

* This capacity is not fully obtained by the dam; the major part of it is the natural capacity of a lake; Owen Falls is not the greatest man-made lake.

** C indicates the dam is under construction.

Table 2 (Concluded)

| No. | Capacity $10^6$ $m^3$ | Name | Country | Year |
|---|---|---|---|---|
| 34 | 27,433 | Oahe | USA | 1958 |
| 35 | 26,000 | Razzaza Dyke | Iraq | 1970 |
| 36 | 25,400 | Rybinsk | USSR | 1941 |
| | 24,700 | Longyangxia | China | C** |
| 37 | 24,700 | Mica | Canada | 1972 |
| 38 | 24,000 | Tsimlyansk | USSR | 1952 |
| 39 | 23,700 | Kenney | Canada | 1952 |
| 40 | 23,500 | Ust-Khantaika | USSR | 1970 |
| 41 | 22,950 | Furnas | Brazil | 1963 |
| 42 | 22,119 | Fort Peck | USA | 1937 |
| 43 | 21,626 | Xinanjiang | China | 1960 |
| 44 | 21,166 | Ilha Solteira | Brazil | 1973 |

48. Table 3 shows the distribution of dams by type and height. Most dams are of the embankment type, most being earthfill but some rockfill. For heights above 60 m, there are more concrete gravity and arch dams than embankments. The number of dams decrease with height. Eighty percent of the dams have heights in the range of 15 to 30 m. About one percent have heights greater than 100 m.

Table 3

Distribution of Dams by Type and Height*

| Dam Height, m | Type of Dam: Embankment | Gravity | Arch | Buttress | Multarch | Total |
|---|---|---|---|---|---|---|
| 15-30 | 24,567 | 2,222 | 775 | 175 | 74 | 27,813 |
| 30-60 | 3,657 | 1,294 | 428 | 110 | 48 | 5,537 |
| 60-100 | 477 | 361 | 204 | 40 | 13 | 1,095 |
| 100-150 | 116 | 65 | 83 | 12 | -- | 276 |
| 150-200 | 21 | 8 | 24 | -- | -- | 53 |
| 200 | 6 | 4 | 13 | -- | 1 | 24 |
| Total | 28,844 | 3,954 | 1,527 | 337 | 136 | 34,798 |

SOURCE: 1984 World Register of Dams
* ICOLD member countries only.

## Geographical distribution of large dams

49. The distribution of large dams amongst continents is shown in Table 4 for the years 1950 and 1982. The 23 countries which have more than 100 dams in 1982 are listed in Table 5. These 23 countries had 95 percent of the world's dams in 1982 and 94 percent in 1950. Outside of China, 11,015 dams were constructed during the period 1950 through 1982, which is more than a threefold increase. With only 8 dams in 1950, the Chinese constructed 18,587 additional dams by 1982. Thus, the Chinese constructed almost 1.7 times as many dams during the 32-year period as the remainder of the world combined.

50. Over one-half of the large dams in the world are located in China. Practically all the Chinese dams are of the embankment types, and 80 percent are less than 30 m in height. The highest dam in China is the 165-m-high Wujiangdu Dam which is an arch-gravity structure.

51. Comparing Tables 1 and 2 with Table 4 shows that the extremely large dams and reservoirs are not located in the countries with the most numerous projects. Of the world's 25 highest dams, 14 are located in the 23 countries with more than 100 dams. Only 6 of the 25 largest reservoirs are located in the 23 countries listed in Table 4. Russia is the leading country in terms of the largest projects in the world.

52. The 353 dams with heights greater than 100 m are distributed among 45 countries. However, two-thirds of these dams are located in 10 countries: United States of America (USA), Japan, Spain, Switzerland, Italy, Canada, France, Russia, China, and India.

Table 4

Countries With More Than 100 Dams*

| Country | Number of Dams In 1950 | Number of Dams In 1982 |
|---|---|---|
| China | 8 | 18,595 |
| USA | 1,543 | 5,338 |
| Japan | 1,173 | 2,142 |
| India | 202 | 1,085 |
| Spain | 205 | 690 |
| Korea | 116 | 628 |
| Canada | 189 | 580 |
| Great Britain | 378 | 529 |
| Brazil | 142 | 489 |
| Mexico | 109 | 487 |
| France | 164 | 432 |
| Italy | 199 | 408 |
| Australia | 122 | 374 |
| South Africa | 79 | 342 |
| Norway | 48 | 219 |
| Germany | 46 | 184 |
| Czechoslovakia | 44 | 142 |
| Sweden | 32 | 134 |
| Switzerland | 35 | 130 |
| Yugoslavia | 12 | 114 |
| Austria | 20 | 112 |
| Bulgaria | 4 | 108 |
| Romania | 6 | 106 |
| Total | 4,876 | 33,368 |

SOURCE: 1984 World Register Dams
* ICOLD member countries only.

Table 5

Distribution of Dams by Continent*

| Continent | Number of Dams In 1950 | | In 1982 | |
|---|---|---|---|---|
| Asia | 1,541 | 29.7% | 22,701 | 65.2% |
| Americas | 2,090 | 40.2% | 7,241 | 20.8% |
| Europe | 1,293 | 24.9% | 3,800 | 10.9% |
| Africa | 123 | 2.4% | 610 | 1.8% |
| Australia | 150 | 2.9% | 446 | 1.3% |
| Total | 5,196 | 100.0% | 34,798 | 100.0% |

SOURCE: 1984 World Register of Dams
* ICOLD member countries only.

## PART III: DAMS AND APPURTENANT STRUCTURES

53. A reservoir project includes various water control structures. Spillways allow floodwaters to be discharged while preventing damage to the dam. Outlet works regulate the release or withdrawal of water for beneficial purposes. Water may be released to the river below the dam or withdrawn from the reservoir to be conveyed by pipeline or canal to the location where it is used. Hydroelectric power plants require appurtenant water control facilities. Navigation locks may be included in a dam to facilitate river transport. The configuration of the dam and appurtenant structures is unique for each project. However, general characteristics of typical types of structures are described in the following paragraphs. In-depth treatments of dam and appurtenant structure design are provided by the US Bureau of Reclamation (1976, 1977a, 1977b), Thomas (1976), Golze (1977), and Davis and Sorensen (1984).

### Dam Types and Configurations

54. Although timber, steel, and stone masonry have been used in constructing dams, most dams are earthfill, rockfill, or concrete. Dams constructed of natural excavated materials placed without addition of binding material are termed embankments (Figure 1). As further illustrated in Figure 1, concrete dams may be categorized as gravity, arch, or buttress. The stability of a gravity dam is derived primarily from its weight. Arch and buttress designs reduce the amount of concrete required to withstand the forces acting on a dam. Arch dams transmit most of the horizontal thrust of the water stored behind them to the abutments and have thinner cross sections than gravity dams. A buttress dam consists of a watertight upstream face supported on the downstream side by a series of intermittent supports termed buttresses.

55. As indicated in Table 3, the 34,798 large dams included in the World Register of Dams (ICOLD 1984) are distributed among the different types as follows: earthfill and rockfill, 83 percent; gravity, 11 percent; arch, 4 percent; buttress, 1 percent; and multiple arch, 1 percent. The 353 dams with heights greater than 100 m are distributed as follows: earthfill and rockfill, 41 percent; gravity, 22 percent; arch 34 percent; buttress, 3 percent; and multiple arch 0.3 percent. Almost all of the embankment dams are

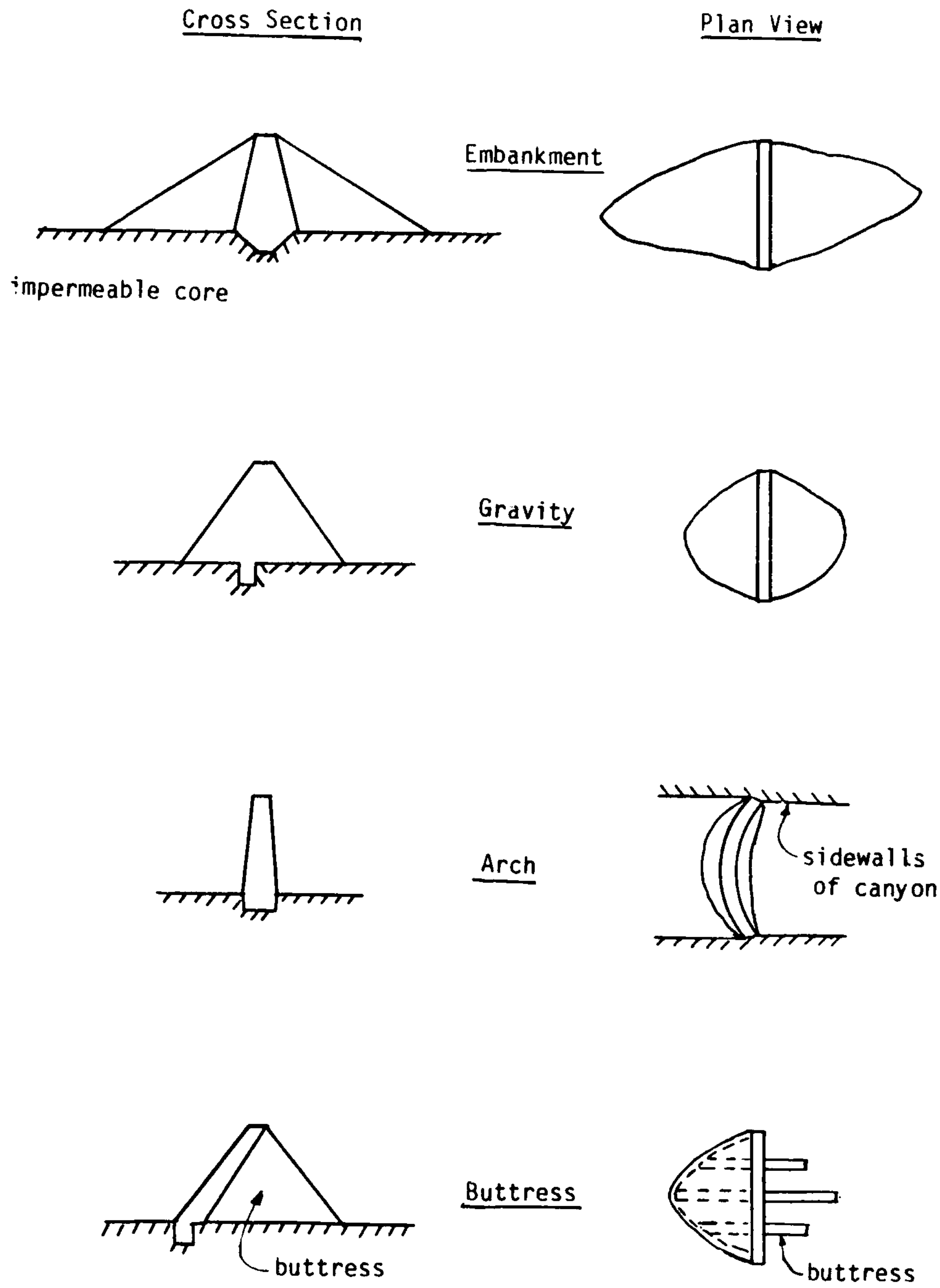



Figure 1. General types of dams

earthfill, the remainder being rockfill or some combination of earthfill and rockfill.

56. Dams are also classified as overflow or nonoverflow. Overflow dams are designed for water to flow over their crests. Nonoverflow dams are designed to not be overtopped. Overflow dams are limited essentially to concrete. Earthfill and rockfill dams are damaged by the erosive action of overflowing water, and consequently, supplemental concrete structures are required to serve as spillways.

57. More than one type of dam may be included in a single structure. For example, a concrete section may contain a spillway, with the remainder of the dam being an earthfill embankment. Curved dams may combine both gravity and arch effects to achieve stability.

Embankment dams

58. Earthfill is the most common type of dam because materials are usually available at the construction site and foundation requirements are generally less stringent than for other dam types. In the past, a number of earthfill dams were constructed by hydraulic-fill methods involving the use of water to transport and place the material in the dam. In modern practice, the rolled-fill method has been used almost exclusively. Selected materials are placed in layers and compacted with a heavy roller.

59. Earthfill dams are of three types: homogeneous, zoned, and diaphragm. A homogeneous dam is composed of essentially the same material throughout. The material must be sufficiently impervious to provide an adequate water barrier, and the slopes must be relatively flat for stability. Although small dams were often constructed in this manner in the past, few large dams are homogeneous.

60. Commonly, embankment dams are zoned as illustrated by Figure 1. Zoned embankments have a central impervious core, a transition zone along both faces of the core, and outer zones of more pervious material for stability and protection of the impervious core. The impervious zone may consist of clay or a mixture of clay, silt, and sand. The pervious zones may consist of sand, gravel, cobbles, or rock, or mixtures of these materials.

61. Diaphragm-type dams have a thin diaphragm of concrete, steel, timber, or earth which serves as a water barrier with the bulk of the embankment constructed of pervious material such as sand, gravel, or rock. In the case of an earth diaphragm, this type dam is differentiated from a zoned embankment by the relative thickness of the impervious barrier. The position of the

diaphragm may vary from a blanket on the upstream face to a central vertical core.

62. Earthfilled dams typically have a rock layer protecting the slopes from erosion. Rock toes are often used for drainage of the embankment.

63. The bulk of a rockfill dam is composed of rocks of various sizes which provide the stability for the structure. An impervious membrane is required to make the dam watertight. This membrane may be an upstream face of concrete, asphalt, impervious earth, or other material, or a core of impervious soil near the center of the dam. There is no clear-cut distinction between earthfill and rockfill dams. Some dams are composed of a combination of both types of material. Typically, rockfill dams are designed with steeper slopes than earthfill dams.

Gravity dams

64. Gravity dams are generally constructed of stone blocks or concrete. However, gravity dams of uncemented stone were constructed several thousand years B.C. With the passing of the centuries, various types of mortar were used to bind the stones together, thereby increasing permissible slopes, stability, and watertightness. Modern gravity dams are primarily of mass concrete, and each dam relies on its own weight for stability. The dam is usually roughly triangular in cross section with its base width so related to its height as to ensure stability against overturning, sliding, or foundation crushing. Most gravity dams are straight in plan, but some are slightly curved.

Arch dams

65. An arch dam is curved in plan and carries most of the water load horizontally to the abutments by arch action. Consequently, these dams are typically found in steep canyons with sidewalls capable of resisting the arch forces. Although arch dams were probably constructed earlier, the oldest known arch dam was built on the Turkish-Syrian border during the period 527-565 A.D. (Schnitter 1967). Many early arch dams were constructed of rubble masonry, but practically all of these dams constructed in recent years are of concrete.

Buttress dams

66. A buttress dam consists of a sloping membrane which transmits the water load to a series of supporting members, called buttresses, at right angles to the axis of the dam. Buttress dams have been constructed in various configurations. Typical types include the flat slab and multiple arch. The

face of a flat slab buttress dam is a series of flat reinforced-concrete slabs. The face of a multiple-arch dam consists of a series of arches which permit wider spacing of buttresses. Buttress dams usually require only one-third to one-half as much concrete as gravity dams of similar height. Consequently, buttress dams may be used on foundations which are too weak to support a gravity dam.

67. The Meer Allum Dam, near Hyderabad, India, built over 100 years ago, was possibly the first multiple arch buttress dam (Smith 1971). The first reinforced concrete slab buttress dam was built in the United States by Nils Ambursen in 1903. This type of dam is often called an Ambursen dam.

## Spillways

68. A spillway is a safety valve for a dam. Spillways provide a means for releasing floodwaters or other inflows in excess of normal storage and outlet capacities. The excess water is drawn from the top of the impounded pool and conveyed through a spillway structure and appurtenant channel to the river below the dam. A spillway may be used to allow normal riverflows to pass over, through, or around the dam whenever the reservoir is full. Spillways also protect the dam from extreme flood events. Spillway capacity is a critical factor in dam safety, particularly for embankment dams which are likely to be destroyed if overtopped.

69. A spillway may be controlled or uncontrolled. A controlled spillway has gates which can be used to adjust the flow rate. Many reservoirs have a single spillway. Some reservoirs have two or more spillways, a service spillway to convey frequently occurring overflows and one or more emergency spillways used only during extreme flood events. For some reservoir configurations, water flows through the spillway a large portion of the time; in other cases, the spillway is designed to be used only for an extreme flood event expected to occur possibly once in several hundred years.

### Spillway types

70. A variety of configurations have been adopted in spillway design. Spillways may be categorized by the path the water takes, i.e. over, through, or around the dam. Typical types include overflow, chute, side-channel, shaft, and siphon spillways.

71. Overflow spillway. An overflow spillway is a section of dam designed to permit water to flow over the crest. In some cases, the entire

length of the dam is an overflow spillway. Overflow spillways are widely used on concrete gravity, arch, and buttress dams. Some earthfill dams have a concrete gravity section designed to serve as an overflow spillway.

72. Chute spillway. A spillway in which water flows from an upstream river location to a downstream river location through an open channel, located either along a dam abutment or through a saddle some distance from the dam, is called a chute, open channel, or trough type spillway. The chute spillway has been used more often than any other type of spillway with earthfill dams. The chute may be paved with concrete or asphalt. In some cases where the spillway is expected to be rarely needed, an unpaved chute through a saddle may be used, realizing that some erosion damage will result whenever the infrequent flood does occur.

73. Side-channel spillway. In a side-channel spillway, water flows over the crest into an open channel running parallel to the crest. The crest is usually a concrete gravity section, but it may consist of pavement laid on an earth embankment or the natural ground surface. This type of spillway is used in narrow canyons where sufficient crest length is not available for overflow or chute spillways.

74. Shaft spillway. In a shaft spillway, the water drops through a vertical or inclined shaft to a horizontal conduit or tunnel under, around, or through the dam. Shaft spillways are often used where there is inadequate space for other types of spillways. The inlet may consist of a square-edged lip. A shaft spillway with an inlet curved to increase the hydraulic efficiency is often called a "morning glory" spillway.

75. Siphon spillway. A siphon spillway consists of a closed conduit in the shape of an inverted U, with the bend serving as the spillway crest set at the normal water surface elevation. The inlet is generally placed well below the normal reservoir water surface to prevent entrance of ice and debris and to avoid formation of vortices and drawdowns which might break the siphon action. At low flows, the siphon spillway hydraulically operates like an overflow spillway. After the conduit in the bend fills, the spillway becomes a siphon. The primary advantage of a siphon spillway is the capability to maintain a constant full-capacity discharge rate.

Spillway shapes

76. A spillway control section may be a simple flat, broad-crested weir or, alternatively, may be curved to increase the hydraulic efficiency. The ogee-shaped spillway crest has a curved profile designed to approximate the

shape of the lower nappe of a ventilated sheet falling from a sharp-crested weir. At the design head, water flows smoothly over the crest with little resistance from the concrete surface, thus maximizing the discharge. The profile below the upper curve of the ogee spillway is continued tangent along a slope, often with a reverse curve at the bottom of the slope directing the flow onto the apron of a stilling basin.

Spillway components

77. Spillways typically include an entrance structure or overflow crest, discharge channel or conduit, terminal structure, and approach and outlet channels.

78. In some situation, such as the case of an overflow spillway over a concrete dam, approach and outlet channels may not be required. The water flows directly from the reservoir over the spillway to the river below. However, in many cases, channels are provided to direct the flow to the spillway entrance structure and to convey the flow from the terminal structure back to the river.

79. Water is conveyed from the entrance structure over, around, under, or through the dam to the terminal structure in channels, conduits, or tunnels. As previously discussed, spillways can be classified based on the conveyance method. However, a few spillways have no conveyance structure. For example, the discharge may fall freely through the air from an arch dam crest, or flow may be released directly along an abutment to cascade down the hillside.

80. The difference in elevation between the reservoir water surface and downstream river results in extremely high flow velocities at the spillway exit. Consequently, energy dissipation is usually required to prevent damaging erosion. A principle function of a terminal structure is to dissipate kinetic energy prior to release of the water to the outlet channel or river. Concrete stilling basins are typically provided to facilitate loss of energy in the turbulence of a hydraulic jump. Baffle blocks and end sills increase the efficiency of the energy loss in the basin. Other types of terminal devices include deflector buckets where flow is projected as a free-discharging upturned jet to fall into the stream channel some distance below the end of the spillway. Erosion in the streambed may be minimized by fanning the jet into a thin sheet by the use of a flaring deflector.

Spillway crest gate types

81. An ungated or free overflow spillway crest automatically regulates the discharge as a function of the elevation of the reservoir water surface, without release decisions by an operator being required. Additional control of the storage capacity above the spillway crest can be provided by crest gates. The full-discharge capacity of the spillway may be utilized during extreme flood events with water being stored behind closed gates during non-flooding or less severe flooding situations. Many types of spillway crest gates have been devised. Several common types are illustrated in Figure 2 and described in the following paragraphs.

82. Tainter gates. The tainter, or radial, gate is probably the most widely used type of crest gate for large installations. Tainter gates are usually constructed of steel or a combination of steel and wood. The cylindrical face of the gate is supported by radial arms attached to trunnions set in the downstream portion of the piers on the spillway crest. The gate pivots around the trunnions as it is opened or closed. Water flows between the bottom of the gate and the spillway crest when the gate is raised. Flexible fabric or rubber stripping is used to form a water seal between the gate and the piers and spillway crest.

83. The gate face is made concentric to the pivot pins so that the entire force of the water passes through the pins. Thus, the moment required to be overcome in raising and lowering the gate is minimized. Counterweights are often used to partially counterbalance the weight of the gate and thus reduce the required capacity of the hoist. The small hoisting effort needed to operate tainter gates makes hand operation practical on small installations. However, gates are typically operated by cables fixed to motor-driven winches set on platforms above the gate.

84. Tainter gates vary in size from 1 to over 10 m in height and from 2 to 20 m in width. A spillway may contain as many as 20 or more gates set side by side. Each gate may have its independent hoisting mechanism or a common unit may be moved from gate to gate.

85. Due to the relatively small hoisting forces involved, tainter gates are more adaptable than other types of gates to operation by automatic control apparatus. Multiple gates can be arranged to open automatically at successively increasing reservoir levels. Some gates may be opened automatically with the remaining gates on the spillway requiring manual operation.

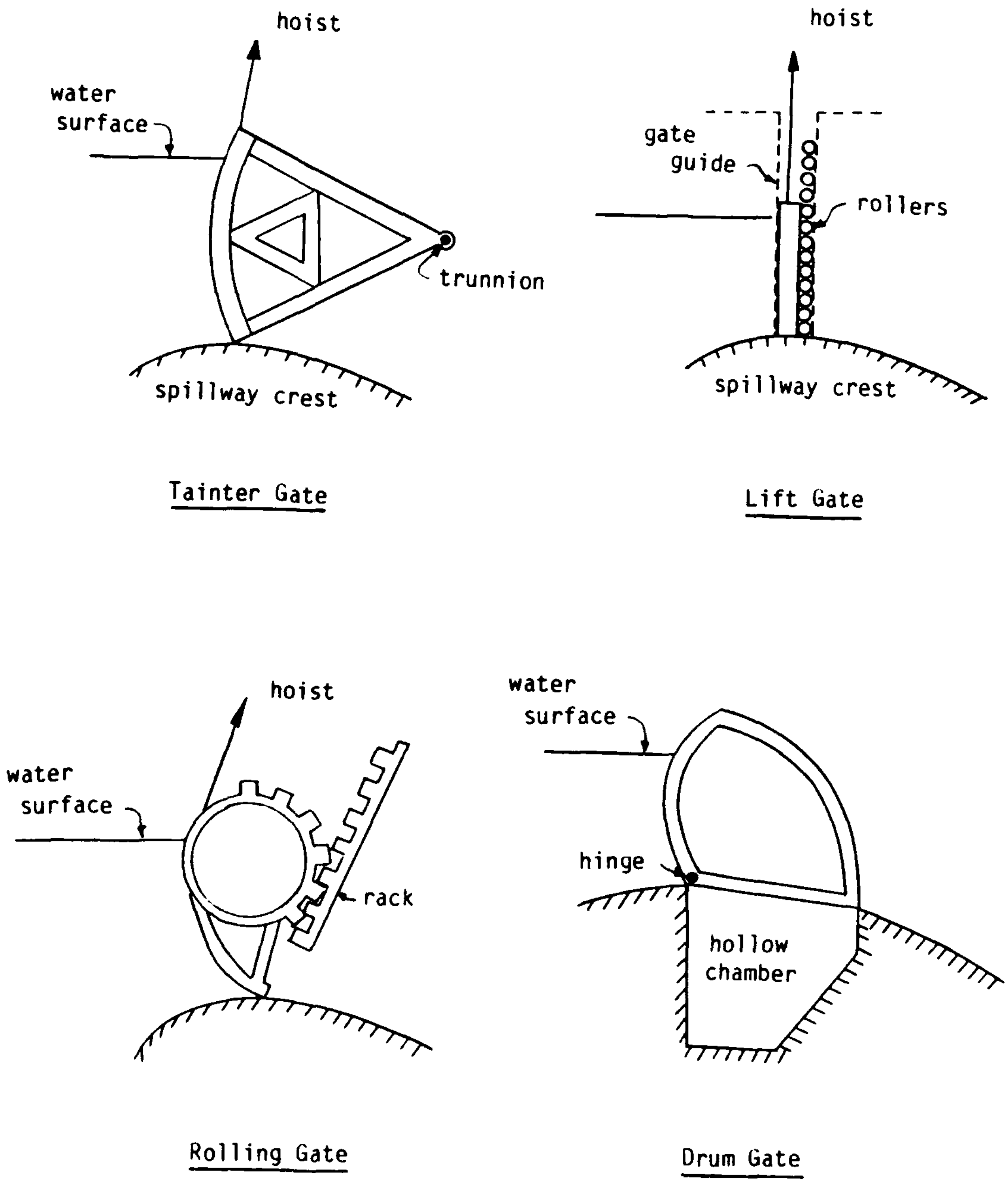



Figure 2. Types of spillway crest gates

86. Lift gates. Rectangular lift gates span horizontally between guide grooves in supporting piers. The support guides may be either vertical or inclined slightly downstream. The gates are raised or lowered by an overhead hoist. Water flows over the spillway crest, under the opened gate. The gates are typically made of steel but at some dams are timber or concrete.

87. The edges of a lift gate may bear directly on the supporting guides. However, the sliding friction that must be overcome to operate the gate limits the gate size for which this type of installation is practical. Rollers or wheels are often used to reduce the frictional resistance and thereby permit use of a larger gate and/or smaller hoist. Large lift gates are often built in two horizontal sections so that the upper portion may be lifted and removed from the guides before the lower portion is moved. This design reduces the load on the hoisting mechanism and minimized the headroom required.

88. Rolling gates. A rolling, or roller, gate consists of a steel cylinder spanning between the piers. Each pier has an inclined rack which engages gear teeth encircling the ends of the cylinder. The gate is rolled up the rack with a cable and hoist, allowing water to flow beneath the gate. Rolling gates are well adapted to long spans of moderate height.

89. Drum gates. A drum gate consists of a hollow drum which, in the lowered or open position, fits in a recess in the top of the spillway. When water flows over the spillway crest and into the recess, the gate is lifted, completely or a least partially, by the buoyant force.

90. Stop logs. Stop logs are sometimes used as an economical substitute for more elaborate gates where relatively close spacing of piers is not objectionable and gate openings are required only infrequently. Stop logs are horizontal beams or girders set one upon the other to form a bulkhead supported in grooves in piers at each end of the span. Discharge is controlled by installing and removing stop logs. The logs may be raised by hand or with a hoist.

## Outlet Works

91. Whereas spillways are provided to handle floods and other inflows surpassing the reservoir storage capacity, an outlet works is used for normal project operations. The outlet works controls the storage capacity below the spillway crest elevation. Releases are made to meet municipal, industrial,

and agricultural water supply needs; to maintain flows in the river downstream for navigation, pollution abatement, and preservation of aquatic life; and for other beneficial purposes. An outlet works also serves to empty the reservoir to allow inspection, maintenance, and repairs to the dam and other structures. In addition, the outlet works may be used for flood control, to evacuate storage below the spillway crest in anticipation of flood inflows, or to supplement spillway releases during and after a flood event.

92. At some dams, an outlet works has been combined with a service spillway and used in conjunction with a secondary emergency spillway. In this situation, the usual outlet works design is modified to include an overflow weir which automatically bypasses surplus inflows whenever the reservoir rises above the normal storage level. Extreme flood events exceeding the capacity of the combined service spillway and outlet works are handled by a separate emergency spillway.

93. In many cases, the outlet works empties into the river channel below the dam. The water may serve instream purposes and/or be withdrawn from the river at some distance below the dam. In other cases, the outlet works discharges directly into a canal or pipe conveyance system for transport to the location of water use.

Outlet works components

94. An outlet works typically consists of a sluiceway, intake structure, gates or valves, terminal structure, and entrance and exit channels.

95. Sluiceways. A sluiceway is a passageway through, under, or around a dam. Sluiceways for concrete dams generally pass through the dam. Often the outlet conduit is placed through a spillway overflow section, using a common stilling basin to dissipate energy for both spillway and outlet works flows. For embankment dams, the sluiceway is typically placed outside the limits of the embankment fill material. If a conduit is placed through an embankment, collars are normally used to reduce seepage along the outside of the conduit. Sluiceways are typically concrete, though steel or other materials may be used. Tunnels through rock abutments are sometimes constructed without lining. Cross sections may be circular or rectangular. In large concrete dams, multiple smaller conduits are often used instead of a single large conduit. A penstock is a sluiceway designed to carry water to hydroelectric turbines.

96. Intake structures. Although the entrance to a sluiceway may be an integral part of the dam or another structure, most outlet works have an

intake structure. The primary function of the intake structure is to permit withdrawal of water from the reservoir over a range of pool levels and to protect the conduit from damage or clogging as a result of waves, currents, debris, or ice. Intake structures vary from a simple concrete block supporting the end of a pipe to elaborate concrete intake towers.

97. An intake structure may be either submerged or extended as a tower to some height above the maximum reservoir water surface, depending on its function. A submerged intake consists of a rock-filled crib or concrete block which supports the end of the conduit. Submerged intakes are widely used on small projects because of their low cost.

98. A tower is required if operating gates are located at the intake or if a platform is needed for installing stop logs or maintaining and cleaning trashracks and fish screens. Intake towers are usually provided with ports at various levels which may aid flow regulation and permit some selection of the quality of water to be withdrawn. A wet intake tower consists of a concrete shell filled with water to the level of the reservoir and has a vertical shaft inside connected to the withdrawal conduit. Gates are normally provided on the inside shaft to regulate flow. With a dry intake tower, the entry ports are connected directly to the sluiceway, without water entering the tower.

99. Intake structures are often provided with trashracks to prevent entrance of debris. Trashrack structures can be found in various designs and configurations. The racks usually consist of steel bars spaced several centimeters apart. Debris accumulations may be removed by hand or by automatic power-driven rack rakes. Screens are also sometimes provided to prevent fish from being carried through the outlet works.

Gates and valves

100. Intake structures usually contain control devices. In some cases, normal flow regulation is achieved by gates or valves at the intake. In other cases, flow is regulated by gates or valves located in the sluiceway some distance downstream of the entrance. However, additional gates are still provided in the intake structure to dewater the conduit for inspections or repairs. A valve in the interior of the sluiceway may be used to regulate flow, with intake gates being used routinely to keep the sluiceway empty during periods of no releases.

101. Entrance gates. Gates at the sluiceway entrance are often used to regulate flow for projects with heads less than roughly 30 m. For higher

heads, because of cavitation and vibration problems associated with partly opened gates under high heads, entrance gates are usually used only to dewater the sluiceway for maintenance and repair of the conduit or downstream gates. Small gates on low-head installations are often simple sliding gates operated by hand- or motor-powered drives. Slide gates often have bronze bearing surfaces to minimize friction. Rollers are required for high-head installations or for very large gates under low heads.

102. Tractor gates are often used for outlet works under high heads. A tractor gate is rectangular in shape and lifts vertically in grooves. Wedge-shaped roller trains are attached to the back of the gate on either side. As the gate is lowered into the closed position, its downward motion is halted when its bottom edge comes in contact with the bottom of the gate frame. The roller trains, moving in slots beside the gate, continue their downward movement, and because of their wedge shape, permit the gate to move a small distance downstream. The pressure of the water forces the gate tightly into the gate frame to form a watertight seal. Air ducts are sometimes provided in the sluiceway to reduce cavitation during gate operation. Hoisting equipment is located above the gate.

103. Ports in wet intake towers are typically controlled by gates mounted either inside or outside the shaft. The gates consist of a steel plate and framework which can be raised or lowered to cover the port opening.

104. Bulkheads and stop logs are often provided for dewatering the sluiceway and possibly the intake tower for maintenance and repairs. Bulkhead slots may be provided in the intake structure with the bulkheads being hoisted into place when needed.

105. Interior gate valves. At many dams, releases are regulated by valves located in the sluiceway at some distance downstream of the entrance. For sluiceways in gravity dams, the valve operating mechanism is often in a gallery inside the dam. In other cases, the operating mechanism extends to the surface of the dam. For heads under 25 m, flow is often regulated by gate settings. For greater heads, gates are ordinarily used in only the fully-open or fully-closed position. High-head regulating valves, such as needle and Howell-Bunger valves, allow varying valve settings. Multiple sluices allow discharge rates to be varied by the number of sluices open.

## Other Structures

106. Other water control structures associated with dams include water supply intake and diversion structures, hydroelectric power plants, and navigation locks.

Water supply diversions

107. Water for agricultural, municipal, industrial, and other uses may be withdrawn directly from the reservoir or from the river at some distance below the dam. Intake towers with pumps may be located near the dam or in the upper reaches of a reservoir. Water is pumped from the reservoir to be conveyed by pipeline to the location where it is used. In other cases, water released through an outlet works is pumped from the river at downstream locations.

108. The term "barrage" is sometimes used to refer to relatively low-head diversion dams often associated with irrigation. The function of a barrage is to raise the river level sufficiently to divert flow into a water supply canal.

Hydroelectric power plants

109. Each hydroelectric power project has its own unique layout and design. The powerhouse may be located at one end of the dam, directly downstream from the dam, or between buttresses in a buttress dam. In some cases, water is conveyed through a penstock to a powerhouse located some distance below the dam. With favorable topography, a high head can be achieved in this manner even with a low dam. A reregulating dam is often provided below the hydroelectric plant.

110. A hydroelectric power project typically includes, in some form, a diversion and intake structure, a penstock or conduit to convey the water from the reservoir to the turbines, the turbines and governors, housing for the equipment, transformers, and transmission lines to distribution centers. A forebay or surge tank regulates the head. Trashracks and gates are typically provided in the intake structure. A draft tube delivers the water from the turbines to the tailrace, through which it is returned to the river.

Navigation locks

111. Dams on rivers used for navigation often include locks. A navigation lock is a rectangular boxlike structure with gates at either end that allows vessels to move upstream or downstream through a dam. Lockage occurs as follows, assuming a vessel is traveling upstream. The lock chamber is

emptied. The downstream gate is opened and the vessel enters the lock. The chamber is filled, with the water lifting the vessel to the level of the reservoir. The upstream gate is opened and the vessel departs. A lock at the Ust-Kamengorsk Dam on the Irtish River in the Union of Socialist Soviet Republics has a lift of 42 m. The highest lock in the United States is the John Day lock on the Columbia River at 34.5 m (Linsley and Franzini 1979).

## PART IV: RESERVOIR OPERATION

112. The following discussion focuses on how reservoir storage capacity is beneficially utilized for various project purposes. Reservoir operation is addressed from the perspective of procedures and practices followed in determining the quantity of water to store and to release or withdraw under various conditions. Each reservoir or reservoir system is unique with its own project purposes and operating policies. However, the basic concepts outlined here are generally representative of reservoir operation throughout the world.

### Reservoir Pools

113. Reservoir release policies or operating procedures are often based on dividing the total storage capacity into designated pools. A typical reservoir consists of one or more of the vertical zones, or pools, illustrated by Figure 3.

114. Water releases or withdrawals are normally not made from the inactive pool, except through the natural processes of evaporation and seepage. The inactive pool is sometimes called dead storage. The inactive pool may provide sediment reserve, head for hydroelectric power, and water for recreation. The top elevation of the inactive pool may be fixed by the invert of the lowest outlet or, in the case of hydroelectric power, by conditions of operating efficiency for the turbines.

115. The conservation pool supplies water for various beneficial uses. The reservoir water surface is maintained at or as near the top of the conservation pool elevation as streamflows and water demands allow. Drawdowns are made as required to meet water supply needs. Reservoir operation strategies may include designation of one or more buffer zones. Full demands are met as long as the reservoir water surface is above the top of the buffer zone, with certain nonessential demands being curtailed whenever the water in storage falls below this level.

116. The flood control pool remains empty except during and immediately following a flood event. The top elevation of the flood control pool may be set by the crest of an uncontrolled spillway. Gates allow the top of the flood control pool to exceed the spillway crest elevation. For the common case of a reservoir with no designated flood control capacity, the top of the

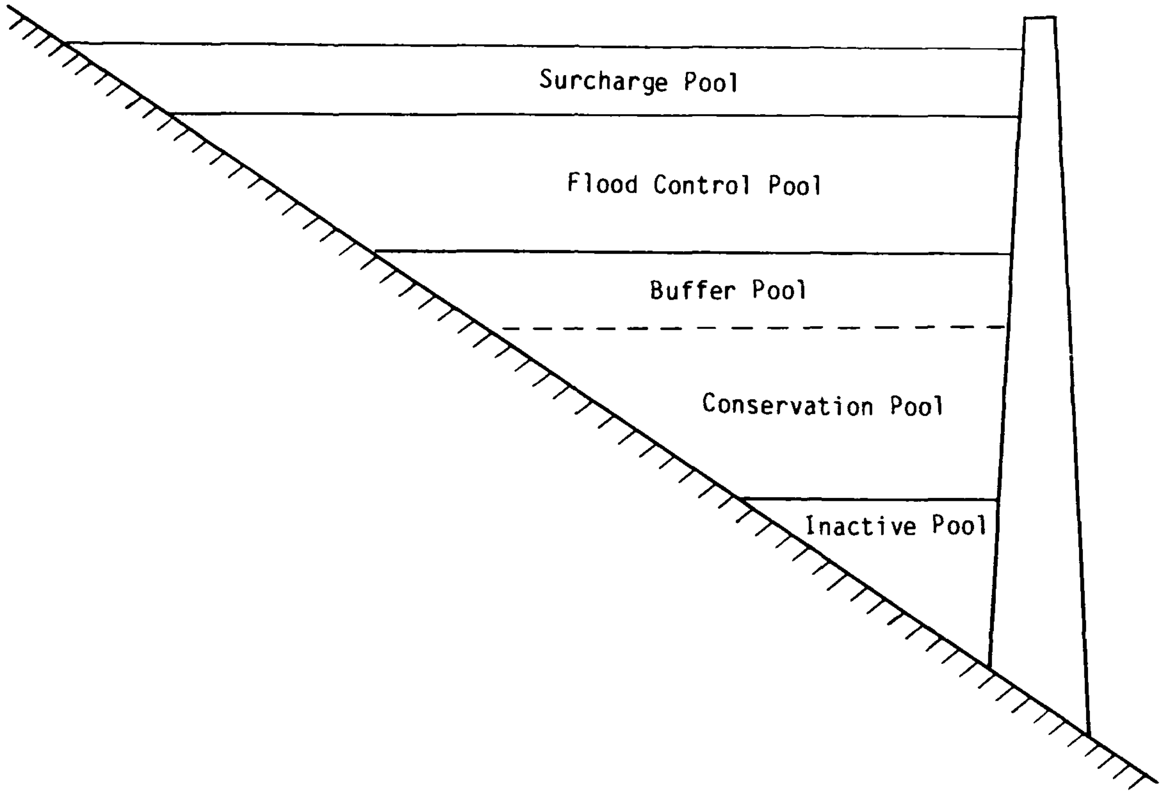



Figure 3. Reservoir Pools

conservation pool is often fixed by the elevation of an uncontrolled spillway crest.

117. The surcharge pool is uncontrolled storage above the conservation and/or flood control pools which occurs during a flood as inflow to a full reservoir exceeds outflow. The maximum design water surface is an elevation established during project design from the perspective of dam safety. The structural integrity of the dam could be threatened if the surcharge storage exceeds the maximum design water surface. Consequently, assuring that the reservoir water surface does not exceed the maximum design water surface is an important consideration in reservoir operation.

## Multiple-Purpose Reservoir Operation

118. Reservoirs are operated for single or multiple purposes which include municipal, industrial, and agricultural water supply; cooling water for steam-electric power plants; hydroelectric power generation; navigation; fisheries; recreation; and flood control. Maintenance of instream flows for

fish and wildlife habitat, dilution of pollution, aesthetics, and freshwater inflows to bays and estuaries are often significant considerations in reservoir operation also. Additionally, dams have also been constructed for the purpose of disposal of mine tailings, salt brine, and various other potential pollutants. Dams located near the coast can serve as a barrier against saltwater encroachment.

119. Project purposes can be categorized as (a) conservation storage, (b) diversion, (c) flood control, and (d) other miscellaneous purposes. Conservation storage evens out variations in streamflow and water demands over time to facilitate the beneficial use of the water. Conservation purposes can be divided between uses which require withdrawal of water from the reservoir-stream system (such as municipal, industrial, and agricultural water supply) and instream uses (such as hydroelectric power, navigation, and recreation). Diversion dams provide head to facilitate diversion of water from the river by pumps or gravity flow, but generally contain negligible storage capacity. Flood control involves temporarily storing floodwaters to reduce downstream damages. Disposal of mine tailings and prevention of saltwater encroachment are examples of purposes which fall in the category of other miscellaneous purposes.

120. Although single-purpose reservoirs are not uncommon, major reservoir systems typically serve multiple purposes. Many of the complexities associated with reservoir operation involve allocation of limited storage capacity and water to competing purposes and users. Although often conflicting, the various purposes are sometimes complementary. For example, reservoir releases for municipal, industrial, or agricultural water supply may be routed through hydroelectric power turbines before being withdrawn from the river at downstream locations. Water stored for water supply purposes provides an excellent opportunity for recreation as well as additional head for hydroelectric power.

121. Multiple-purpose operations are often based on designated reservoir pools. Reservoir release decisions are dependent on reservoir water-surface elevation. For example, reservoir operation may be based on the conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing floodwaters to reduce downstream damages. As illustrated in Figure 3, flood control and conservation pools are fixed by a designated top elevation of the conservation pool. The pool above this elevation is maintained empty

except during and immediately following major flood events. Operating policies are based on keeping the conservation pool as full as inflows and demands will allow. Likewise, a portion of a conservation pool may be designated as the hydroelectric power pool. Hydroelectric power is generated only if water is in the power pool. If the reservoir water surface falls below this pool, municipal water supply withdrawals may continue while hydroelectric power releases are curtailed.

122. Water availability, water supply demands, hydroelectric power demands, risk of flooding, and flood damage susceptibility vary seasonally. In many parts of the world, most of the annual rainfall or snow melt occurs during a certain period of the year. Floods often tend to occur during a distinct season. Thus, streamflow may be highly seasonal. Likewise, water demands vary during the year. Agricultural water demands depend on the irrigation season. The extent of agricultural flood damage also depends on whether the flood occurs during the growing season. Seasonal-rule curve reservoir operations are frequently adopted in response to seasonally varying conditions. A seasonal-rule curve consists of varying the top elevation of the conservation pool, or other pool, as a function of the time of year.

## Operation for Conservation Purposes

123. Development and management of water resources involves modification of the hydrologic cycle to regulate the natural water supply to better meet human needs. Precipitation does not occur at the optimal times and places to meet human needs. Excessive amounts of precipitation flow back to the ocean and may even cause damaging floods at some locations and times while severe shortages of water occur at other times and places. The purpose of reservoirs is to alter the temporal and spatial distribution of the runoff resulting from precipitation to better conform to the needs of society. Reservoirs are much more effective at altering temporal than spatial runoff distribution. However, combined with conveyance facilities, reservoirs also are used to transport runoff from one basin to another where it is needed.

124. Conservation storage increases the dependability of a surface water resource. Streamflow can be beneficially used without a reservoir. However, natural streamflow variability frequently means that water needs can be met only a portion of the time. During dry periods, sufficient water will likely not be available to satisfy all needs. Constructing a dam to store

high flows increases the likelihood of maintaining adequate streamflows to meet needs during dry periods. Similarly, for a given storage capacity, increased withdrawals from the reservoir result in an increased risk for future water shortages.

125. In general, reservoir operation can be categorized as being primarily influenced by either seasonal fluctuations in streamflow and/or water use or long-term threat of drought. In many parts of the world, a reservoir will be filled during a distinct season of high rainfall or snow melt and emptied during a dry season with high water demands. Thus, the reservoir level fluctuates greatly each year in a predictable seasonal cycle. In other reservoirs, much of the conservation storage is provided as protection against the long-term threat of a severe drought. Water is stored through many wet years to be available during drought conditions. In this case, a drought is characterized as a series of several dry years rather than the dry season of a single year.

126. Reservoir operations are governed by institutional considerations. Limited water resources are allocated to competing users within the framework of political and economic systems. The water needs to be met and operating policies to be followed depend upon the entities which own and operate a reservoir system and the entities which hold the right to the use of the water. Water law of a country or region controls the legal rights to the use of water. Reservoirs are operated in accordance with contractual arrangements between reservoir management organizations and water users. Management of major reservoir systems may also involve agreements between nations. Throughout the world, some 214 river basins are shared by two or more countries (United Nations 1978). These basins comprise about 50 percent of the land area of the world.

127. Water resources development purposes and associated structures and facilities are described by Linsley and Franzini (1979) and Viessman and Welty (1985). Water is utilized for a variety of purposes. Some types of use involve withdrawal of water from the stream system, typically with a portion of the water being returned after use. Other uses are made of the water in a reservoir or stream without withdrawal. Several major uses of conservation storage are cited below.

<u>Water supply</u>

128. Reservoir operation procedures for municipal, industrial, and agricultural water supply are based on meeting demands, subject to

institutional constraints related to water right , contractual arrangements, and governmental agreements. Water supply withdrawals are made at many projects through pumping plants with intake structures located in the reservoir. In many other cases, releases are made through outlet works and spillway structures to be withdrawn from the river at downstream diversion and intake facilities. Water may be actually withdrawn at locations hundreds of river miles below the dam from which it was released. Diversion dams are often constructed at point of diversion to provide head to facilitate withdrawal of water from the river. Conveyance facilities may be used to transport water significant distances from the point of diversion to the location at which the water is actually used.

129. Many water supply reservoirs are operated as individual units to supply specific users. Other reservoirs are operated as components of a multiple-reservoir system. System operation may involve maintaining a balance between storage depletions and water-surface fluctuations in the component reservoirs. Conjunctive management of surface water reservoirs and ground-water aquifers may be advantageous under appropriate circumstances. Likewise, demand management can be integrated with supply management. As reservoir storage is depleted, a water demand reduction can be enforced and a greater reliance can be placed on ground-water supplies. As previously discussed, buffer zone operations provide a mechanism for reducing reservoir withdrawals as storage is depleted.

130. Most water supplied for irrigation is typically lost through evapotranspiration. A significant portion of municipal and industrial water supply withdrawals may be returned to the stream as wastewater. In highly urbanized areas, wastewater may be a significant portion of the total inflow to a reservoir.

131. Steam-electric power plants require large volumes of water for condenser cooling, but most of the water is returned to the reservoir or stream. The water removing the heat from the generating plant may be circulated through evaporative cooling towers or other cooling devices or returned directly back to the reservoir or the stream below the reservoir. Cooling water reservoirs are normally maintained at a relatively constant pool level.

Hydroelectric power

132. Hydroelectric plants are generally used to complement the other components of an overall power system. Because the demand for power varies over the course of a day (or other time period), the terms base load and peak

load are commonly used to refer to the constant minimum power demand and the highest instantaneous power demand, respectively. Hydroelectric power is typically used to meet peak load requirements while thermal plants supply the base load. Hydroelectric power plants can assume load rapidly and are very efficient for meeting peak demand power needs. In some regions of the world, hydroelectric power is the primary source of electricity, supplying most of the base load as well as peak load. Availability of water is generally a limiting factor in hydroelectric energy generation.

133. Firm or dependable power is the output that a plant can essentially provide all the time. Firm power is based on critical low streamflow or other minimum conditions of water availability. Surplus or secondary power is all power available in excess of firm power. Firm power is typically much more valuable than secondary power.

134. Hydroelectric plants may be classified as run-of-river, storage, or pumped-storage. A storage-type plant has a reservoir with sufficient capacity to permit carry-over storage from the wet season to the dry season or from wet years through a drought. The plant could be at a reservoir, or storage could be provided by one or more upstream reservoirs. The conservation storage capacity increases the firm power. A run-of-river plant has very limited storage, and flows through the turbines are essentially limited to natural streamflow. A run-of-river plant may have enough storage, called pondage, to permit storing water during off-peak hours for use during peak hours of the same day. On the other hand, a run-of-river plant may have a significant amount of inactive storage which provides head but not water for release through the turbines. A pumped-storage plant generates energy for peak load, but at off-peak, water is pumped from the tailwater pool to the headwater pool for future use. The pumps are powered with secondary power from some other plant in the system, often a run-of-river plant where water would otherwise be wasted over the spillway.

135. At many projects, reservoir releases are made specifically and only to generate hydroelectric power. At other projects, hydroelectric power generation is limited essentially to releases which are being made for other purposes, such as municipal, industrial, or agriculture water supply. An upstream reservoir may be operated strictly for hydropower, with the releases being reregulated by a downstream reservoir for water supply purposes.

Navigation

136. The three basic approaches for improving a river for navigation are channel improvements, canalization, and lock-and-dams. Channel improvements involve dredging, straightening, snag removal, contraction works, and bank stabilization. Canalization involves construction of a totally new channel around an obstruction or between two navigable waters. Upstream reservoirs can be used in combination with improved channels and canals. Reservoir releases can be made during periods of low natural flow to maintain minimum flow depths. However, large quantities of water are typically required to augment riverflows for navigation. Consequently, reservoir storage has not been widely used to maintain downstream flows for navigation.

137. Lock-and-dam operations are common in major waterways through the world. Dams create a series of slack-water pools with sufficient depths for navigation. The dams are equipped with locks to lift or lower barges and other vessels from one pool to the next.

Other instream flow needs

138. In addition to those discussed above, instream flow needs include maintenance of sufficient streamflow for water quality, fish and wildlife habitat, freshwater inflows required to support estuarine ecosystems, livestock water, river recreation, and aesthetics. Releases for hydroelectric power and water supply which are withdrawn from the river at significant distances below the dam contribute to instream environmental needs as well. Operating procedures for some reservoirs include releases specifically for maintenance of minimum instream flow levels. Reservoirs can be provided with multilevel outlet works to allow selective blending of discharge waters for optimal downstream water quality.

## Flood Control Operations

139. Construction of a conservation reservoir can actually worsen downstream flooding conditions because of lost valley storage, decreased flood wave attenuation, and increased travel time. However, conservation capacity provides some incidental flood protection when a flood event coincides with a partially drawndown pool. Surcharge storage in conservation only reservoirs may also provide some incidental flood protection. Likewise, temporary storage of floodwater in flood control pools may provide some incidental benefits for conservation purposes.

140. The following discussion addresses designated flood control storage capacity. A reservoir may be operated specifically and only for flood control, or a flood control pool may be included in a multiple-purpose reservoir. Whereas a conservation pool is kept as near full as possible to have water available when needed, a flood control pool is normally kept empty.

141. Flood control reservoirs have either controlled or uncontrolled outlet structures. Uncontrolled spillways are designed with limited discharge capacities so that outflow rates are less than inflow rates and reservoir storage occurs during a flood. An uncontrolled reservoir reduces the peak of the flood hydrograph automatically without release decisions by an operator. Controlled outlet structures have gates. Outflow is a function of gate openings as well as water-surface elevation. Thus, the operator controls release rates by manipulating gate openings. Gated outlet works and spillways provide flexibility for more effective utilization of flood control storage capacity. Most larger projects have gated outlet structures. Uncontrolled outlet structures are used primarily for relatively small flood retarding dams on minor tributary streams.

142. Flood control operations are unique to each specific reservoir or reservoir system. However, the basic procedures followed by the USACE are representative of flood control operations in general. Guidelines for developing flood control operating plans as outlined by the USACE (1959a) are summarized below.

143. The overall strategy for operating the gates of a flood control reservoir consists of two sets of procedures, regular and alternative. The set of procedures requiring the largest release rate controls the given flooding and storage conditions. The regular procedure, which usually controls, is based on the assumption that ample storage capacity is available to handle the flood without special precautions being necessary to prevent the water surface from rising above the top of the flood control pool. Operation is switched to the alternative schedule during extreme flooding conditions when the predicted runoff from a storm would exceed the controlled capacity remaining in the reservoir. If the water-surface level significantly exceeds the top of the flood control pool, downstream damages will necessarily occur. The objective is to assure that reservoir releases do not contribute to downstream damages as long as the storage capacity is not exceeded. However, for extreme flood events which would exceed the reservoir storage capacity, moderately high damaging discharge rates beginning before the flood control is full are

considered preferable to waiting until a full reservoir necessitates much higher release rates.

144. An example regulation schedule is presented in Figure 4 (USACE 1959a). The reservoir release rate is read directly from the graphs. The schedule is repeated in two formats, labeled schedule A and schedule B. Using schedule A, release decisions are based on a current water-surface elevation and inflow rate. With schedule B, release rates are dependent upon the current water-surface elevation and rate of rise of water surface. The two forms of the schedule are intended to result in the same release rate. Schedule A is used if measured inflow rates are known. In the absence of measured inflow rates, schedule B is used based on rate of rise of water-surface elevation. Release rates are typically determined at a reservoir control center which has access to real-time streamflow measurements. If communications between the control center and operator at the project are interrupted during a flood emergency, the operator can determine gate releases based on schedule B without needing measurements of inflow rates.

145. Downstream flooding conditions are not reflected in the family of curves illustrated in Figure 4. These curves are intended to guide operations only if the regular operating procedure would result in overtopping the flood control pool. The regular procedure is based on not making releases which would contribute to downstream flooding. Releases are not made unless downstream flows are below damaging levels. The regular procedure could be followed until the flood control pool fills. However, after the flood control pool is full, tremendously high discharge rates may be required to prevent the surcharge storage from exceeding the design water surface. The much higher peak release rate necessitated by this hypothetical operation policy can be expected to be much more damaging than a lower release rate with a longer duration beginning before the flood control pool is full. On the other hand, an operator would not want to make damaging releases early in a storm if the flood control pool remained empty during the storm. Although expected streamflows that will occur several hours or days in the future are sometimes incorporated in real-time operations, forecast flows are still highly uncertain.

146. The regulation schedule curves are developed based on estimating the minimum volume of inflow that can be expected in a flood, given the current inflow rate and reservoir elevation. After the minimum inflow volume to be expected during the remainder of the flood is estimated, the outflow required to limit storage to the available capacity is determined by mass

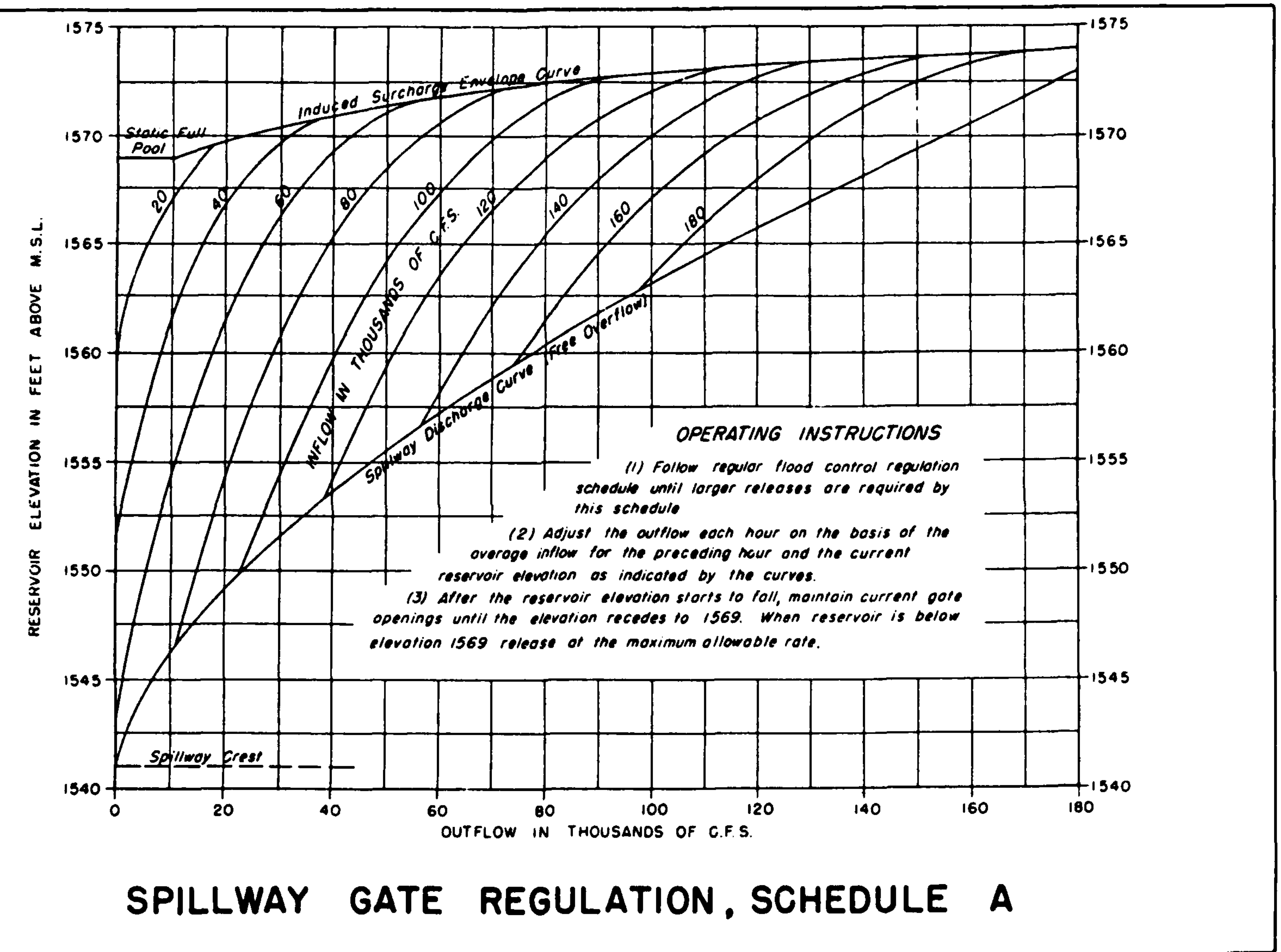
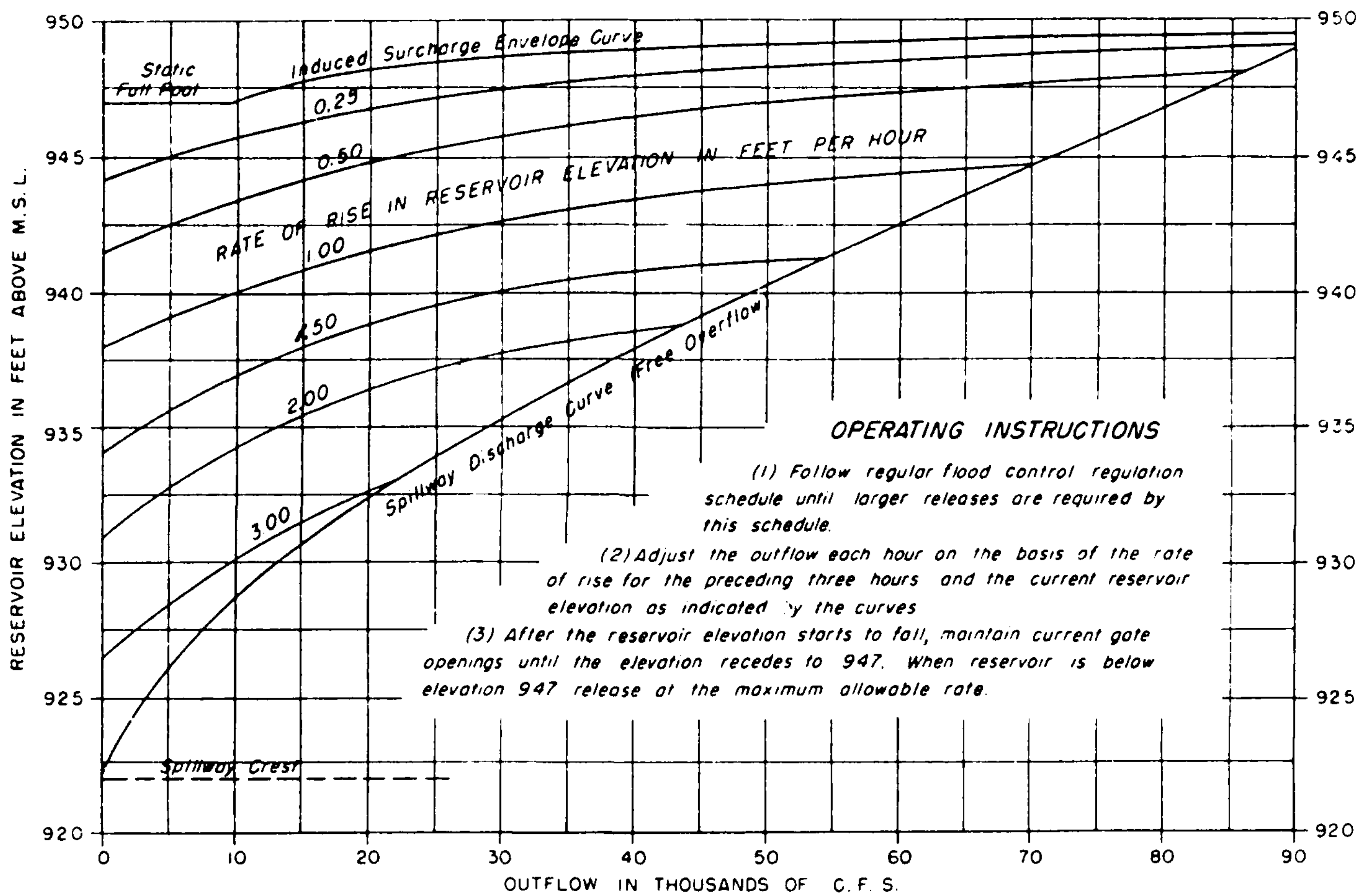



Figure 4. Flood control regulation schedule (Continued)




**SPILLWAY GATE REGULATION, SCHEDULE B**

Figure 4. (Concluded)

balance computations. For a given current inflow rate, the minimum inflow volume for the remainder of the storm is obtained by assuming the inflow hydrograph has just crested and computing the volume under the recession side of the hydrograph. For conservatively low inflow volume estimates, the assumed recessive curve is made somewhat steeper than the average observed recession. The complete regulation schedule which allows the outflow to be adjusted on the basis of the current inflow and empty storage space remaining in the reservoir is developed by making a series of computations with various assumed values of inflows and amounts of remaining storage available.

147. As previously indicated, the flood control regulation strategy for a reservoir actually consists of two procedures. The regular procedure is followed as long as the indicated releases are greater than the outflow values read from the curves discussed above. The regular schedule is based on downstream flooding conditions. Nondamaging flow rates and stages are specified at selected index locations, called control points, which are representative of the damage potential in the associated reach of channel and flood-plain. Also, nondamaging flow rates are equal to or closely related to bank-full stream capacities. Stream gaging stations are located at the control points. Releases are made to empty the flood control pool as quickly as possible without exceeding the allowable flow rates at each downstream control point. The regulation schedule consists of specified flow rates to be maintained at the designated control points.

148. When a flood occurs, the spillway and outlet works gates are closed. The gates remain closed until a determination is made that the flood has crested and flows are below the nondamaging levels specified for each of the control points. The gates are then operated to empty the flood control pool as quickly as possible without exceeding the allowable flows at the control points. Normally, no flood control releases are made if the reservoir level is at or below the top of the conservation pool. However, if flood forecasts indicate that the inflow volume will exceed the available conservation storage, flood control releases from the conservation storage may be made if downstream conditions permit. The idea is to release some water before the water level rises downstream, if practical for a forecasted flood.

149. For many reservoirs, the allowable flow rate associated with a given control point is constant regardless of the reservoir surface elevation, assuming the outflow still exceeds the value specified by the previously discussed graph illustrated by Figure 4. At other projects, the flood control

pool is subdivided into two or more zones with the allowable flow rates at one or more of the control points varying depending upon the level of the reservoir surface with respect to the alternative zones. Thus, stringently low flow levels can be maintained at certain locations as long as only a relatively small portion of the flood control pool is occupied, with the flows increased to a higher level, at which minor damages could occur, as the reservoir fills. The variation in allowable flow rates at a control point may also be related to whether the reservoir level is rising or falling.

150. A reservoir is operated on the basis of maintaining the flow rates at several control points located various distances below the dam. The most downstream control points may be several hundred miles away. Lateral inflows from uncontrolled watershed areas below the dam increase with distance downstream. Thus, the impact of the reservoir on flood flows decreases with distance downstream. Operating to downstream control points requires streamflow forecasts. Flood attenuation and travel time from the dam to the control point and inflows from watershed areas below the dam must be estimated as an integral part of the reservoir operating procedure.

151. Most flood control reservoirs are components of basin-wide multireservoir systems. Two or more reservoirs located in the same river basin will have a common control point. A reservoir may have one or more associated control points which are influenced only by that reservoir and several others which are influenced by other reservoirs as well. Typically and to the degree practical, reservoirs in a system are operated such that approximately the same percentage of flood control storage is maintained in each reservoir. Releases from all reservoirs, as well as runoff from uncontrolled watershed areas, must be considered in forecasting flows at control points.

152. Maximum allowable rate of change of reservoir releases are also usually specified. Abrupt gate openings causing a flood wave with rapid changes in stage are dangerous and may contribute to streambank erosion.

# PART V: SPILLWAY AND OUTLET WORKS RATING CURVES

153. A rating curve is the relationship between reservoir water-surface elevation and discharge through a spillway or outlet works. Discharge is a function of head above the spillway crest or outlet opening. A family of rating curves is required to express the water-surface elevation versus discharge relationship as a function of gate opening. Rating, or discharge, curves provide fundamental information for real-time reservoir operation as well as for mathematical modeling studies. Since stage is much easier to measure than discharge, the discharge from a reservoir is determined by applying the measured water-surface elevation to the rating curve. For a given measured reservoir level, rating curves are used to select a gate opening or the number of sluices to open to achieve a desired release rate. The computational methods used to develop rating curves for reservoir control structures are outlined in this chapter.

154. Rating curves are developed as an integral part of the design of a reservoir project and are available for operational purposes after completion of construction. Rating curves for existing structures can also be developed from actual measurements of stage and discharge. However, military situations could result in the need to compute rating curves for existing projects with limited data and under expedient conditions.

## References and Computer Programs

155. Procedures followed by the Corps of Engineers in the hydraulic design of spillways and outlet works, including development of rating curves, are outlined in engineer manuals (USACE 1963 and 1965), which rely heavily upon "Hydraulic Design Criteria" prepared for OCE by WES (undated). The US Bureau of Reclamation (1977b) provides another authoritative reference on hydraulic design of spillways and outlet works, which includes empirical coefficients and other data needed for developing rating curves for various types of structures. This general topic area is also included in textbooks and handbooks by Chow (1959) and Davis and Sorensen (1984).

156. The computations involved in developing rating curves can generally be performed manually using a desk calculator. Generalized computer programs are also available. Computer programs available at WES include the following (WES 1985a):

*a*. Stage-Discharge Relation for Standard Ungated Spillways (H1103).

*b*. Stage-Discharge Relation for Unsubmerged Spillway Crests with Uncontrolled Flow (H1105).

*c*. Stage-Discharge Relation for Elliptical Crest Spillways (H1107).

*d*. Calculation of Discharge through a Pressure Conduit Using Darcy-Weisbach Formula (H2030).

*e*. Calculation of Discharge through a Pressure Conduit Using Manning's Equation (H2031).

*f*. Calculation of Discharge through a Horseshoe Conduit Using Manning or Darcy-Weisbach Formula (H2043).

*g*. Calculation of Discharge through a Rectangular Conduit Using Manning or Darcy-Weisbach Formula (H2044).

*h*. Calculation Discharge in an Oblong or Circular Conduit Using Manning or Darcy-Weisbach Formula (H2045).

*i*. Calculation of Outlet Works Loss Coefficient from Prototype Measurement of Drawdown (H2251).

*j*. Stage-Discharge Relation of a Tainter Gate on Curved Crest for Unsubmerged Flow (H3106).

*k*. Vertical Lift Conduit Gates Stage-Discharge Relation (H3201).

## Basic Equations

157. Rating curves are developed using fundamental equations of hydraulics. The equations are covered in standard textbooks and handbooks, such as Brater and King (1976) and Davis and Sorensen (1984), and are reproduced below for ready reference. The application of the basic equations to the specific problem of computing discharges through various types of reservoir control structures is addressed in subsequent sections.

### Continuity and energy equations

158. The continuity equation expresses the concept of conservation of mass. Fluid is neither lost nor gained. For steady, incompressible flow, the continuity equation may be expressed as follows:

$$Q = V_1A_1 = V_2A_2$$

where Q denotes discharge, V is average velocity, and A is a cross-sectional area. The subscripts refer to the location of the cross section.

159. The principle of conservation of energy may be expressed as follows:

$$Z_1 + p_1/\gamma + V_1^2/2g = Z_2 + p_2/\gamma + V_2^2/2g + h_L$$

where Z is the vertical distance above an arbitrary horizontal datum, p is pressure, $\gamma$ is the unit weight of the fluid, V is velocity, g is the gravitational constant, and $h_L$ is head loss. This equation states that the energy at one point in a fluid (subscript 1) is equal to the energy at any downstream location (subscript 2) plus the energy losses occurring between the two locations. The energy is expressed in terms of head, which is energy per unit weight of fluid, with units of foot-pound per pound or metre-newton per newton. Total head is the summation of elevation head (Z), pressure head ($p/\gamma$), and velocity head ($V^2/2g$).

Head loss equations

160. The Manning and Darcy-Weisbach equations are widely used to estimate the head loss ($h_L$) term in the energy equation. The Manning equation is a general purpose formula relating discharge or velocity to channel characteristics for uniform flow. It is also used to estimate head loss for gradually varied flow. Although associated primarily with open-channel flow, the Manning equation can also be applied to pipe flow. The Darcy-Weisbach equation is limited strictly to pipe flow.

161. The Manning equation is as follows:

$$Q = (1.468/n)\ A\ R^{2/3}\ S^{1/2} \qquad \text{(English units)}$$

$$Q = (1/n)\ A\ R^{2/3}\ S^{1/2} \qquad \text{(metric units)}$$

where Q is discharge in cubic feet per second or cubic metres per second, n is an empirically determined roughness coefficient, A is a cross-sectional area in square feet or square metres, R is the hydraulic radius in feet or metres, and S is the slope of the energy line. The hydraulic radius R is equal to A/P, where P is the wetted perimeter. The Manning equation was developed for uniform flow, for which the slope of the energy line (S) is equal to the slope of the water surface (hydraulic grade line in pipe flow) and channel bottom.

Standard hydraulic references, such as Chow (1959), provide empirical data to aid in estimating the roughness coefficient (n).

162. Since $h_L$ is equal to SL, where L is the length of channel or pipe, the Manning equation can be expressed in terms of head loss as follows:

$$h_L = n^2V^2L \;/\; 2.22R^{4/3} \qquad \text{(English units)}$$

$$h_L = n^2V^2L \;/\; R^{4/3} \qquad \text{(metric units)}$$

In gradually varied flow, V and R are estimated as the average of the values at either end of the reach.

163. The head loss due to friction in a straight section of pipe may be estimated by the Darcy-Weisbach equation:

$$h_L = f\ (L/D)\,(V^2/2g)$$

where $h_L$ is head loss, f is an empirical friction factor, L is pipe length, D is pipe diameter, and $V^2/2g$ is velocity head. The friction factor (f) can be determined as a function of pipe diameter, pipe roughness, and velocity using the Moody diagram and accompanying tables found in standard hydraulics references such as Davis and Sorensen (1984) and Linsley and Franzini (1979).

<u>Weir and orifice equations</u>

164. A weir is a notch of regular form through which water flows (Brater and King 1976). An orifice is an opening with closed perimeter and of regular form through which water flows. If the opening flows only partially full, the orifice becomes a weir. A weir and orifice are illustrated by Figure 5.

165. The crest of a weir is the edge or surface over which the water flows. A weir with a sharp upstream corner, or edge, such that the water breaks contact with the crest is called a sharp-crested weir. A broad-crested weir has a horizontal or nearly horizontal crest sufficiently long in the direction of flow so that the nappe will be supported and hydrostatic pressure will be fully developed for at least a short distance. A weir crest may also be rounded.

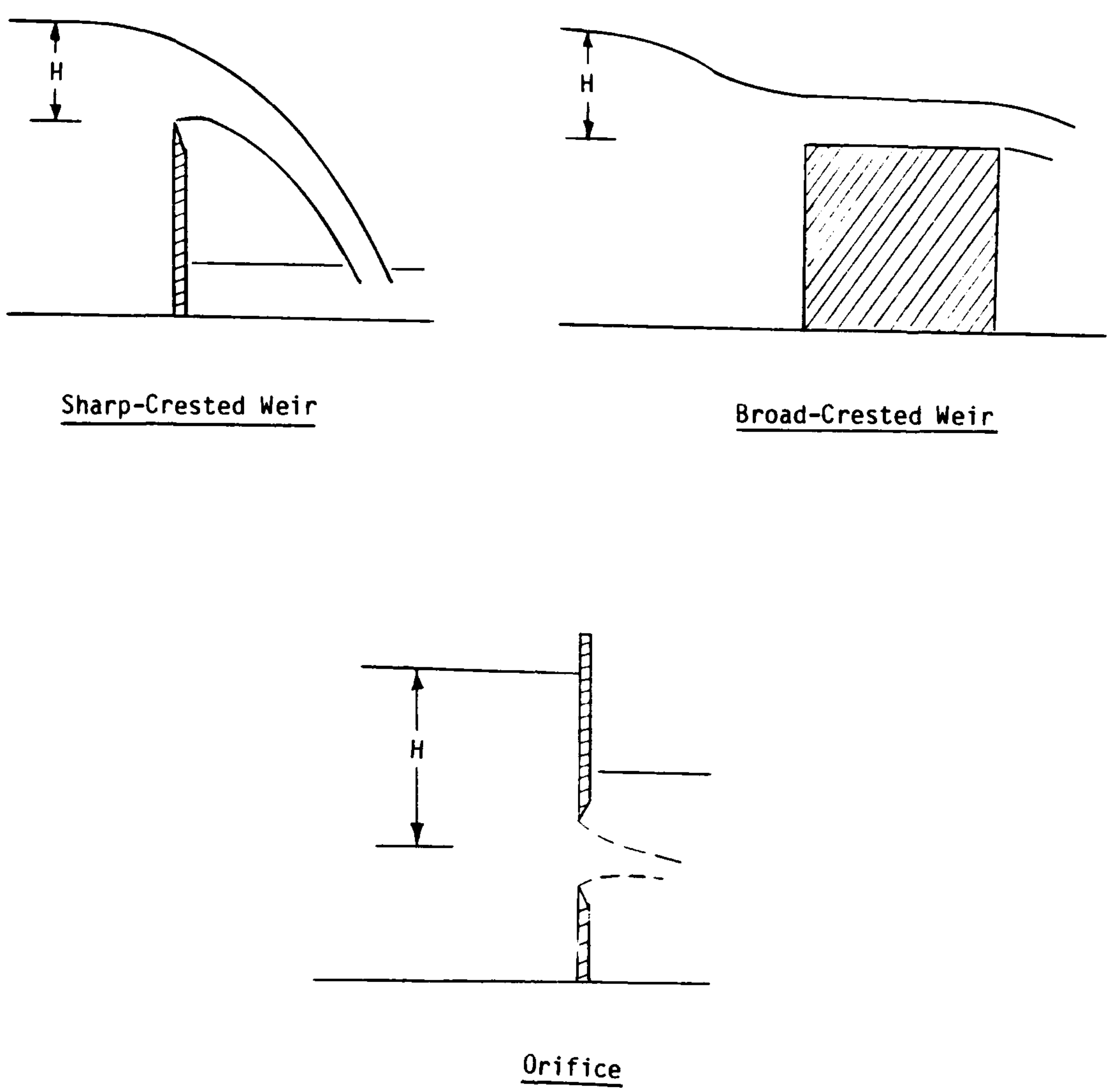



Figure 5. Weirs and orifices

166. The equation for discharge over a weir is as follows:

$$Q = C\,L\,H_e^{1.5}$$

$$H_e = H + V_o^2/2g$$

where Q is discharge, C is an empirically determined coefficient, L is the width of the weir, $H_e$ is energy head above the weir crest, H is head (vertical distance from crest to reservoir surface), $V_o$ is approach velocity, and g is

the gravitational acceleration constant. The discharge coefficient is a function of a number of factors, including the weir shape and configuration, upstream flow conditions, and downstream submergence effects. Assuming L and $H_e$ are expressed in feet and Q in cubic feet per second, the theoretical maximum value of C for a broad-crested weir is 3.087 (Brater and King 1976). Conditions increasing frictional resistance, turbulence, and the resulting energy losses decrease the weir coefficient and corresponding discharge. A rounded-crested weir will be more hydraulically efficient with a larger coefficient than a broad-crested weir, all other conditions being constant. The sharp-crested weir has the largest possible coefficient. Weir coefficients for various conditions are discussed in the following section on discharge over an uncontrolled spillway.

167. When the head is compared with the size of the orifice, the discharge equation is as follows:

$$Q = C\ A\ (2gH)^{0.5}$$

where Q is discharge, C is a discharge coefficient, A is the area of the orifice, and H is head at the center of the orifice. For a sharp-edged circular orifice, C has a value of about 0.60 for a wide range of heads (Brater and King 1976).

168. When the head is compared with the size of the orifice, the discharge for a rectangular orifice is given as follows:

$$Q = 2/3\ C\ (2g)^{0.5}\ L\ (H_2^{3/2} - H_1^{3/2})$$

where L is the orifice width, $H_1$ is head above the top of orifice, and $H_2$ is head above the bottom of orifice. The expression $2/3\ C\ (2g)^{0.5}$ is often designated as a coefficient.

169. Weir and orifice discharge coefficients are typically estimated on the basis of published data, data which have been developed from laboratory and prototype tests. Coefficients for existing reservoir control structures can also be computed directly from discharge versus reservoir drawdown measurements for the particular structure.

## Discharge Over a Spillway

170. The discharge versus head relationship for flow over an uncontrolled spillway is computed using the weir equation noted in paragraph 166.

$$Q = C\ L\ H_e^{1.5}$$

Flow over a spillway crest is a complex phenomenon involving a number of factors. The weir coefficient (C) is a function of head over the crest, downstream submergence conditions, and spillway shape. The effects of abutments and piers on discharge may be taken into account by reducing the net crest length to an effective length (L). Approach velocity is reflected in the energy head ($H_e$).

171. The theoretical maximum value for the weir coefficient (C) of 3.087 is often used for broad-crested spillways. In general, weir coefficients must be estimated on the basis of empirical data derived from prototype or laboratory tests.

### Uncontrolled ogee spillways

172. As previously discussed, the ogee crest approximates the shape of the underside of the nappe of a sharp-crested weir. The ogee shape is commonly used for spillways because it maximizes the discharge for a given crest length and head. Ogee crests are used with overflow, chute, or side-channel spillways, with development of rating curves being essentially the same for the different spillway types.

173. The Corps of Engineers and Bureau of Reclamation have conducted extensive studies on the hydraulics of ogee spillways and have developed standard design methods. The data (USACE 1965) reproduced here is strictly applicable to ogee spillways designed in accordance with Corps of Engineers criteria. However, these and similar available data are also useful, though necessarily approximate, for making estimates of discharges at dams throughout the world even if the exact criteria and methods followed in their design vary from the standard designs for which the data are valid.

174. Discharge coefficients are given as a function of the ratio of head (H) to design head ($H_d$). Figure 6 illustrates the head terms schematically. $H_d$ is set during design, and the shape of the spillway crest is a

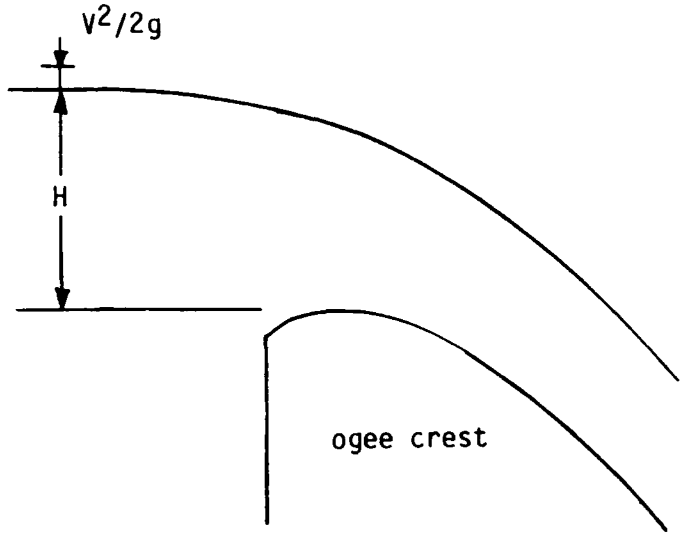



Figure 6. Flow over an ogee spillway crest

function of $H_d$. In many cases, $H_d$ is set to correspond to the maximum reservoir level during the spillway design flood. For reasons of economy, spillway crest shapes have often been designed for a $H_d$ of 75 percent of the head for the maximum reservoir level of the spillway design flood. Hydrologic engineering methods are used to develop the spillway design flood, which typically represents maximum probable flooding conditions.

175. A distinction is made between high overflow spillways which have negligible velocities of approach and low ogee spillways which have velocities of approaches that affect both the shape of the crest and the discharge coefficients. Discharge over a high overflow spillway is also not affected by downstream submergence conditions.

176. With a negligible velocity of approach, the energy head ($H_e$) term in the weir equation becomes simply the head (H). Figure 7 presents values of the discharge coefficient (C) as a function of $H/H_d$ for the standard USACE high-overflow ogee design. C varies from a lower limit of 3.1 for $H/H_d$ of 0.0, 4.03 for $H/H_d$ of 1.0, and 4.13 for $H/H_d$ of 1.33 $H/H_d$ of 1.33 corresponds to the maximum H for the spillway design flood in which $H_d$ is set as 75 percent of the maximum head. The C of 3.1 is comparable to the theoretical value of 3.087 for a broad-crested weir.

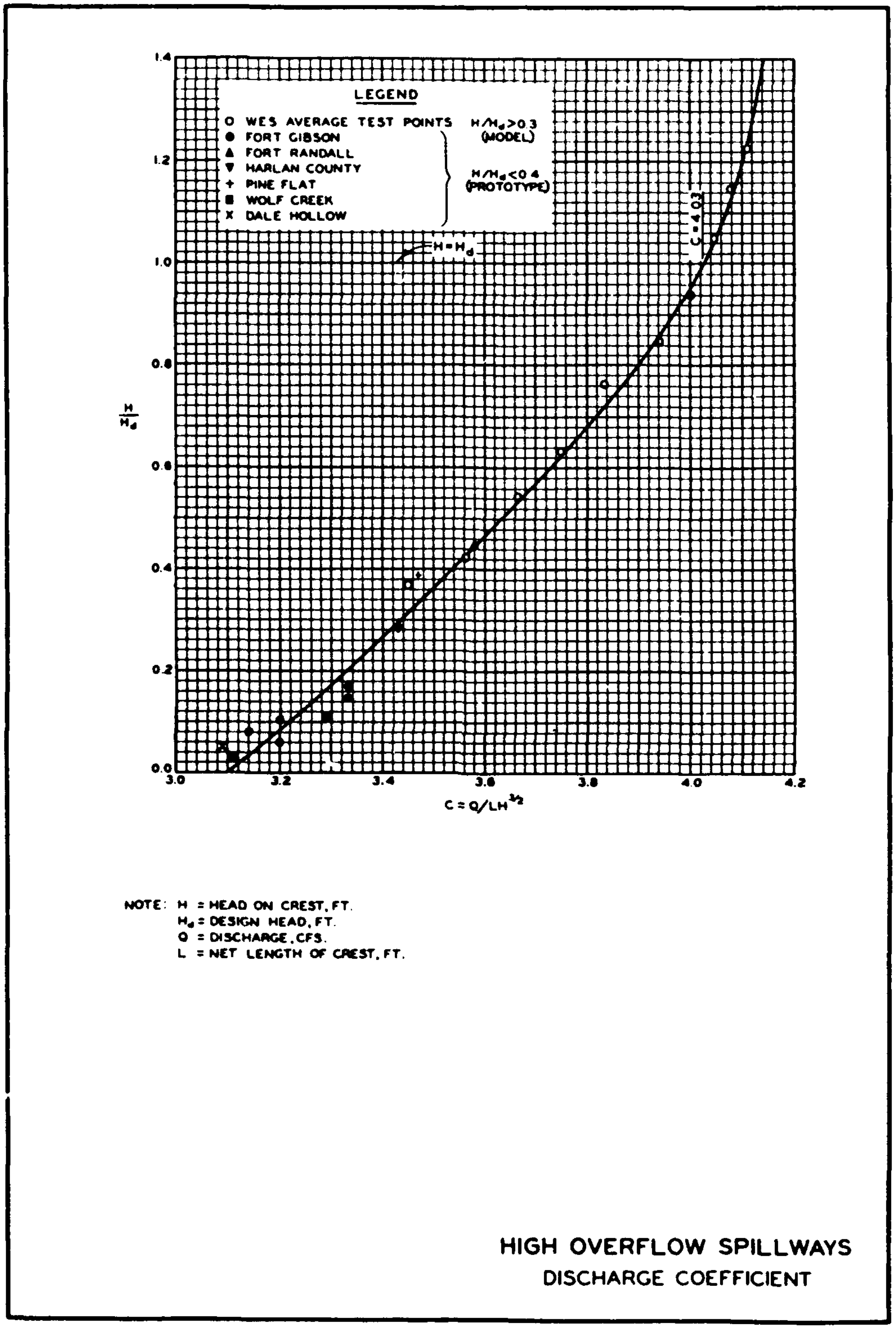



Figure 7. Discharge coefficient

177. The effects of the abutment contraction and piers on the spillway crest can be accounted for by using an effective length (L) in the weir equation determined as follows:

$$L = L' - 2(NKp + K_a)\ H_e$$

where L' is the net length of the spillway excluding the total width of piers, 2 is the number of contractions per gate bay, N is the number of piers, $K_p$ is a pier coefficient, $K_a$ is an abutment coefficient, and $H_e$ is the energy head. Figure 8 presents pier coefficients for several pier shapes. Figure 9 shows abutment contraction coefficients as a function of the radius (R) of abutment rounding.

178. USACE (1965) provides similar information for use in analyzing low overflow spillways. Empirical data for use in correcting the discharge coefficient for submergence effects are also provided. If the spillway crest becomes too deeply submerged, the weir equation is no longer valid, and more complex analysis methods are required.

Gated ogee spillways

179. Rating curves for a gated spillway must be developed as a function of gate opening. For reservoir water-surface elevations above the top of the gate opening, the discharge is computed using the orifice equation. Empirically determined orifice coefficient curves are presented for various types of gates by the USACE (1965) and the US Bureau of Reclamation (1977a,b).

Conduit and shaft spillways

180. Conduits, tunnels, or shafts may flow full or partially full. As discussed in paragraphs 181-188, a spillway with a conduit flowing full is analyzed in the same manner as an outlet works conduit flowing full. With the control at the spillway entrance, the conduit flows partially full. In this case, the weir equation is used to compute discharge. Box-type inlets act as broad-crested weirs. Morning glory spillways consist of a hydraulically efficient rounded overflow crest around a circular vertical entrance shaft. Weir equations and discharge coefficients for these types of spillways are presented by the USACE (1965) and US Bureau of Reclamation (1977a,b).

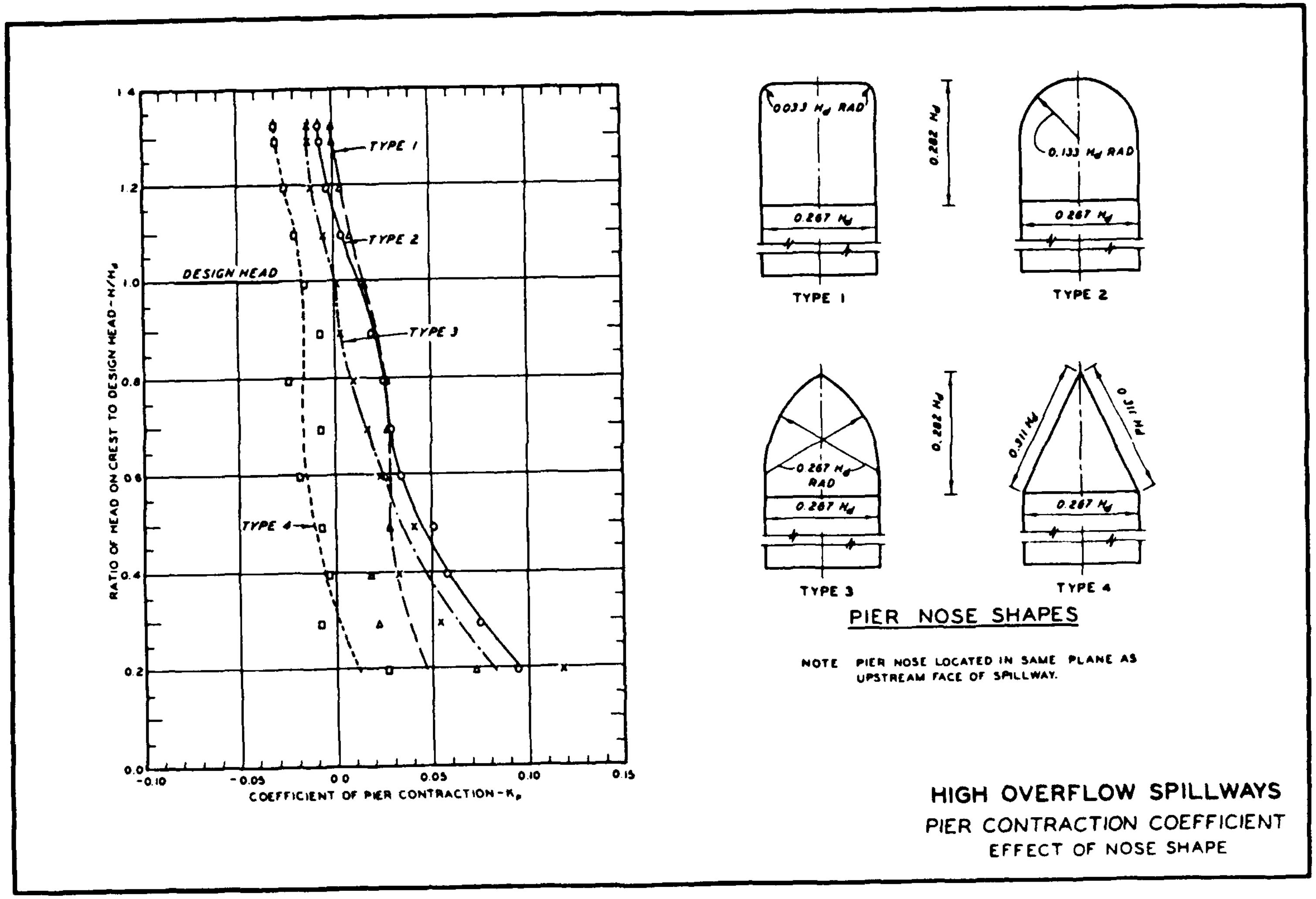



Figure 8. Pier contraction coefficient

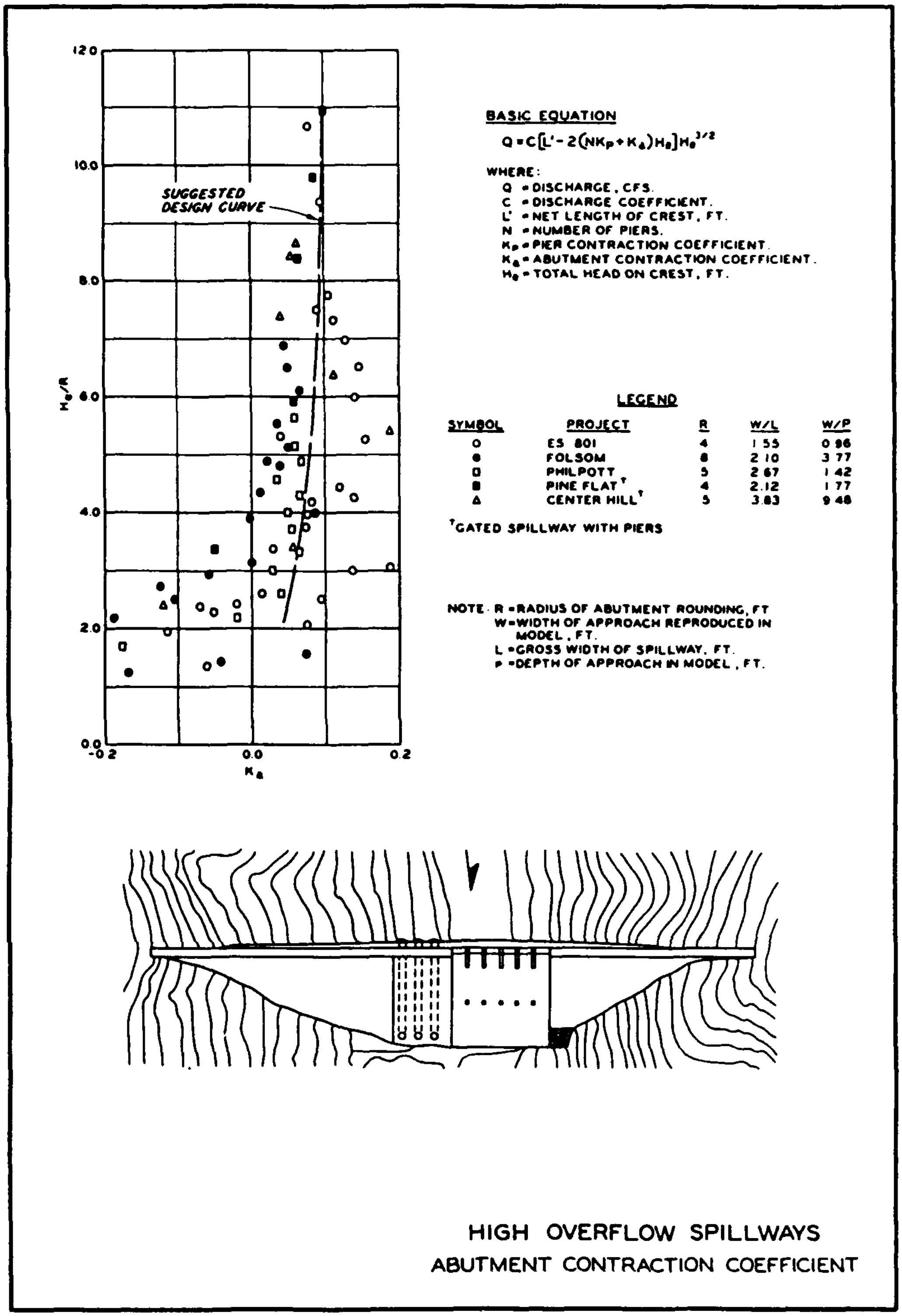



Figure 9. Abutment contraction coefficient

## Discharge Through An Outlet Works

181. Flow through an outlet works occurs with either the sluiceway flowing partially full along its entire length or the sluiceway flowing full along all or a portion of its length. A partially full sluiceway or conduit involves open-channel flow. A full-flowing sluiceway involves pressure conduit flow. The approach for analyzing the discharge versus head relationship depends upon the type of flow. Flow shifts from open channel to pressure conduit as the head increases. Since the exact head at which the switch occurs is difficult to identify, rating curves alternatively assuming open-channel and pressure-conduit flow are typically developed for a transition range of head.

### Partially full conduit

182. The conduit may not completely fill with water because the control is at the entrance. The analysis can then be based on orifice flow. For example, the opening created by a partially- or fully-opened entrance gate can be treated as an orifice. The analysis is conceptually the same as for a gated spillway.

183. For very low reservoir levels, the entrance and conduit may be flowing partially full. If the bottom of the entrance acts as a weir, the discharge can be computed using the weir equation. Otherwise, assuming subcritical flow in the conduit, the reservoir surface elevation versus discharge relationship is developed on the basis of water-surface profile computations. Steady, gradually varied water-surface profile computations are based on the energy equation, with the head-loss term estimated using the Manning equation. The method is described in detail by a number of engineering manuals and textbooks, including USACE (1963), Chow (1959), and Linsley and Franzini (1979).

### Conduit flowing full

184. Resistance to flow, due to frictional effects in the conduit, tailwater, and partially opened downstream valves and gates, results in a conduit flowing full for a given reservoir level and entrance gate opening. Analysis of pressure conduit flow is based on estimating the energy losses caused by each component of the outlet works.

185. The energy equation is applied between the reservoir surface (subscript 1) and outlet exit (subscript 2) as follows:

$$Z_1 + p_1/\gamma + V^2/2g = Z_2 + P_2/\gamma + V^2/2g + h_L$$

$$H + 0 + 0 = 0 + h_{ve} + h_L$$

Thus,

$$H = h_{ve} + H_L$$

where H is the difference in head between the exit and reservoir surface, $h_{ve}$ is the velocity head at the exit, and $h_L$ is total head loss in the system. H is measured from the zero pressure point at the exit. For a free-discharging outlet, this point is the center of the outlet gate opening or uncontrolled exit. For a submerged outlet, H is measured from the surface of the tailwater.

186. Velocities, velocity heads, and head losses are computed to obtain H for a given discharge. The water surface corresponding to the assumed discharge is obtained by adding H to the elevation of the exit zero pressure point.

187. The total head loss ($h_L$) is determined by estimating and summing the head losses caused by various elements of the outlet works structure. The previous equation can be rewritten as follows:

$$H = h_{ve} + h_a + h_b + h_c + \ldots$$

where the subscripts a, b, c refer to head losses associated with individual components of the outlet works.

188. Total head losses typically include trashrack losses, entrance losses, bend losses, contraction losses, expansion losses, gate or valve losses, and conduit friction losses. The Darcy-Weisbach, Manning, or comparable equations are used to determine frictional conduit losses. The head loss caused by various other elements are estimated based on empirical data which are typically expressed in terms of a loss coefficient times a velocity head. Loss coefficients for various outlet works components are presented by the USACE (1963) and Bureau of Reclamation (1977a,b).

## PART VI: MODELING RESERVOIR OPERATIONS

189. Mathematical models, typically in the form of generalized computer programs, provide the capability to analyze reservoir operations from various perspectives. Models provide a quantitative basis for a variety of decisions required in sizing storage capacities and establishing operating procedures for proposed reservoirs during project planning and design; reevaluating operating policies of existing projects in response to changing needs and conditions; evaluating impacts of various actions or activities; and supporting real-time operations. In the past, reservoir operations models have frequently been applied in the civilian sector. Military applications have been few, however, even though the models and basic modeling concepts developed for civilian applications are pertinent.

190. From either a civilian or military perspective, there is no single type of reservoir operation problem but rather a multitude of decision problems and situations. Likewise, a variety of different types of models play various roles in analyzing reservoir operations. Mathematical models used in analyzing reservoir operations can be categorized by function as follows:

- a. Simulation of reservoir operations.
- b. Optimization.
- c. Flood wave analysis.
- d. Watershed modeling.
- e. Synthetic streamflow generation.
- f. Water quality modeling.
- g. Sediment transport modeling.

191. The first category listed consists of models which reproduce the hydrologic and/or economic performance of a reservoir system for given inflows and operating policies. The second category consists of optimization techniques, such as linear programming, dynamic programming, and various other nonlinear programming algorithms, which can be used in determining optimum reservoir-operating policies. Hydrologic and hydraulic routing models are used to compute flow characteristics of a flood wave propagating through a reservoir-stream system as a result of precipitation runoff of reservoir releases. Streamflow hydrographs are fundamental input data for simulating reservoir operations. In the absence of adequate gaged streamflow data, reservoir system inflows are developed by rainfall-runoff (watershed) modeling. Synthetic streamflow generation techniques are also available for

supplementing and extending historical streamflow data. Water quality is also a major concern in planning and operating reservoir systems. The last category listed above is related to the modeling of erosion and sedimentation in a reservoir-stream system.

192. An overview summary of the state of the art of modeling reservoir operations is presented here following the model categorization scheme outlined above. The conceptual basis of each category of models is described, available generalized computer programs are identified, and references providing more in-depth coverage of various topics are cited.

## Simulation of Reservoir Operations

193. A simulation model is a representation of a system used to predict the behavior of the system under a given set of conditions. Simulation is the process of experimenting with a simulation model to analyze the performance of the system under varying conditions, including alternative operating policies. Many types and forms of simulation models have been used for a variety of purposes. Models for simulating reservoir operation typically consist of a collection of mathematical expressions coded for solution on a computer. A reservoir simulation model typically computes storage levels and discharge at pertinent locations in a reservoir-stream system for various sequences of hydrologic inputs (streamflow, precipitation, and evaporation) and demands for releases or withdrawals for various purposes. Physical constraints, such as storage capacities and outlet and conveyance facility capacities, and institutional constraints, such as maintenance of flows associated with downstream water rights, are also reflected in the models. Simulation models also provide the capability to analyze reservoir system operations using hydrologic and economic performance measures such as firm yield, reliability, hydroelectric energy produced, flood damages, and economic benefits associated with various project purposes.

194. Modeling flood control operations is significantly different from modeling reservoir operations for conservation purposes such as municipal, industrial, and agricultural water supplies; hydroelectric power; navigation; recreation; reservoir fisheries; and maintenance of low flows for water quality. Although optional capabilities for analyzing flood control and conservation operations are combined in some models, other models are limited to one or the other type of operation.

Simulation of reservoir operations for conservation purposes

195. A reservoir simulation model is essentially an accounting procedure for tracking the movement of water through a reservoir-stream system. Reservoir releases are determined by the model based on target demands for water supply diversions, instream flows, and/or hydroelectric energy. Diversion and instream flow targets may be specified at downstream control points as well as at the reservoirs. Certain models like HEC-3 and HEC-5, both of which are discussed later, allow diversions and instream flows to be modeled with consideration given to the amount of water in storage. Required demands are met as long as the reservoir storage level is above the top of the inactive pool. Desired demands are met only if the reservoir storage level is above the top of the buffer pool.

196. Modeling reservoir operations are based on a mass balance of reservoir inflows, outflows, and changes in storage, as reflected by the continuity equation:

$$S_2 = S_1 + I - R - E - O$$

where

$S_2$ = reservoir storage at the end of the time period

$S_1$ = reservoir storage at the beginning of the time period

$I$ = reservoir inflows during the time period

$R$ = reservoir releases during the time period

$E$ = evaporation during the time period

$O$ = seepage and other losses during the time period

Seepage and other losses are typically considered negligible. Evaporation is computed by applying an evaporation rate to the average water-surface area during the time period. Thus, a reservoir storage capacity versus water-surface area relationship must be provided as input data. A time series of reservoir inflows and an operating policy for determining releases must be specified.

197. If hydroelectric power is being considered, reservoir storage levels and discharges are converted to electrical power in the model using the power equation:

$$P = \gamma Q h e$$

where

$P$ = power (kilowatts or foot-pounds per second)

$\gamma$ = unit weight of water (kilonewtons per cubic metre or pounds per cubic foot)

$Q$ = discharge (cubic metres per second or cubic feet per second)

$h$ = effective head (metre or feet)

$e$ = efficiency

The effective head (h) is the difference between headwater and tailwater elevations, corrected for hydraulic losses. Tailwater elevation may be expressed as a function of the release rate. The efficiency (e) reflects the power plant energy losses incurred in converting mechanical energy to electrical energy. Energy (kilowatt-hours or foot-pounds) is power multiplied by time.

198. The fundamental mass balance computations performed by a simulation model are essentially the same for either water supply or hydroelectric power. Hydroelectric power simply entails the additional task of relating reservoir water-surface elevation and discharge to power generation for each time interval.

199. Whereas flood control simulation requires a relatively short routing interval to track flood peaks (less than a hour to a day), simulation of conservation operations are typically based on a longer routing interval (such as a month). A simulation may be performed with historical period-of-record, critical period, or synthetically generated streamflows. Period-of-record or average monthly evaporation rates can be used.

200. The information to be obtained from a reservoir simulation will vary depending on the purpose for the study. Model output typically consists of reservoir levels and discharges at the reservoirs and at pertinent downstream locations, as a function of time. System performance in meeting demands can be observed from these output data. A tabulation of reservoir storage levels and discharges may be the only output desired from a simulation. In the case of hydroelectric power, the power produced will be displayed. Firm yield and reservoir reliability can also be determined from simulation studies. Economic as well as hydrologic impacts can be related to discharge and storage levels. A simulation study typically involves numerous runs of a model. A series of runs can be made to compare system performance

for alternative reservoir configurations, operating policies, demand levels, or inflow sequences.

201. Reservoir yield and reliability. Yield is the amount of water which can be supplied from a reservoir in a specified period of time. Quantifying yield is greatly complicated by the stochastic nature of reservoir inflows. Future inflows are unknown and must be estimated based on historical or forecast data. McMahon and Main (1978) provide a comprehensive review of methods for analyzing reservoir yield. Analyses conducted in the planning, design, and operation of reservoirs are typically based on the concept of firm yield. Firm yield is the maximum rate of withdrawal which can be maintained continuously assuming the period-of-record historical inflows are repeated in the future. This yield will just empty the reservoir. Linsley and Franzini (1979) outline the traditional Rippl diagram and sequent peak algorithm approaches for estimating firm yield, which are amenable to manual computations. With the advent of computer simulation models, firm yield is now usually computed by a series of trial simulations. For a given reservoir storage capacity and a given inflow sequence, the system is simulated with alternative trial demand levels in an iterative search for the demand level which just empties the reservoir. The iterative procedure for computing firm yield may be automated within the simulation model.

202. The concept of reservoir reliability expands the concept of firm yield to provide a more meaningful basis for dealing with the uncertainties inherent in the random nature of hydrologic variables. Reservoir reliability is the probability that a specified demand will be met in a given future time period. Reliability is the complement of the risk of failure or probability that the demand will not be met. Reservoir reliability analysis methods typically require inflow sequences many times longer than the period of record. Consequently, synthetic streamflow generation techniques, discussed later in this report, have been developed to provide sufficient data for reservoir reliability studies. Synthetic streamflow generation involves synthesis of equally likely streamflow sequences with a length equal to the time period over which the reservoir is being analyzed. With a large number of equally likely alternative inflow sequences routed through a reservoir using a simulation model, the number of times that demands are met without incurring a shortage due to an empty reservoir can be counted. The reliability is estimated as the percentage of the inflow sequences for which demands are met without incurring a shortage.

203. Firm yield and reliability are discussed above from the perspective of supplying water for various beneficial uses. The concepts are equally applicable to hydroelectric power. Firm power is the maximum rate of energy production which can be maintained continuously assuming the period-of-record historical inflows are repeated in the future. Firm power and reliability associated with various levels of power production are computed with a simulation model similarly to firm yield and reliability for water supply.

204. Economic benefits and losses. Benefits for hydroelectric power can be computed by a reservoir-system simulation model based on primary and secondary energy values, in dollars per unit of energy produced, and the purchase cost for obtaining energy from an alternative source, in case of a shortage in primary energy. Firm energy demands and the associated benefits are provided as input data. Secondary energy is energy in excess of firm energy which is produced by routing releases for other purposes through the turbines. Shortages are computed whenever the firm energy demands can not be met. Cost data are provided as input for assigning dollar losses to shortages.

205. Economic benefits and losses associated with other conservation purposes are typically not included in reservoir simulation models. However, certain models allow shortage versus economic loss functions to be supplied as input data. Economic costs associated with not meeting specified water demands are determined with the model by relating computed water shortages to the input shortage versus loss function.

Simulation of flood-control operations

206. Simulation of flood control operations is based on inflow hydrographs for major flood events, events which could include the probable maximum or standard project floods, hypothetical floods developed from statistical analyses, or historical storms. Whereas simulation of conservation operations typically involve long-term time series of streamflow data, flood control analyses focus on short-time interval data. The primary role of long-term sequences of monthly streamflows in flood control simulation is in determining the reservoir water-surface elevation at the beginning of a flood event.

207. Models for simulating the operation of gated flood control reservoirs include the capability to compute release rates for each time interval during the simulation period based on specified operating rules. Various forms of operating rules may be incorporated into a model. For example, when the water level is in the flood control pool, reservoir releases are typically

based on emptying the pool as quickly as possible without contributing to downstream flooding. Allowable nondamaging discharges are specified at downstream control points. Reservoir inflows and incremental local inflows at the downstream control points are provided as input to the model. For each control point, the model compares the discharge assuming no reservoir release to the allowable discharge. If the allowable discharge is larger, reservoir releases are made. The unregulated flows at a control point may be increased by a contingency factor to compensate for the inability to perfectly forecast flood flows during real-time operations. Since the releases at the reservoir must be routed to the downstream control points to reflect attenuation and travel time, an iterative solution is required to determine the release rate which will maintain the allowable flow levels at the control points. Additional release criteria incorporated into the model include balancing the storage levels in multiple reservoirs releasing to the same control point, and limiting the rate of change of the release rate.

208. Economic evaluation of flood control plans have traditionally been based on the concept of average annual damages. The inundation reduction benefit is defined as the difference in average annual damages without and with a proposed plan. For many years, computing average annual damages using the damage-frequency method described below has been an integral part of the economic evaluation procedures followed by the USACE and other federal agencies in planning flood control improvements. The method is incorporated into several generalized computer programs, including HEC-1 and HEC-5 (Feldman 1981).

209. The magnitude of a flood threat can be quantified in various ways. Discharges, stages, and damages at specified locations can be estimated for historical storms (such as the most severe flood on record), statistical floods (such as the 50- and 100-year recurrence interval floods), and/or hypothetical floods (such as the standard project flood). Expected or average annual damage is actually a frequency-weighted sum of damage for the full range of damaging flood events and can be viewed as what might be expected to occur, on the average, in any present or future year. Additional meaningful information, including discharges, stages, and damages associated with a range of storm magnitudes, are generated in the process of computing average annual damages.

210. A fundamental assumption of the procedure is that damages can be estimated as a function of peak discharge or stage. Additional analyses are

required to show how damages change with variations in flow velocity, duration, and sediment content.

211. Average annual damage computations are based on the statistical concept of expected value. Expected or average annual damage is computed as the integral of the damage versus exceedance frequency function. Exceedance frequency versus peak discharge, discharge versus stage, and stage versus damage relationships are combined to develop the damage versus frequency function.

212. The peak discharge versus exceedance frequency relationship describes the probabilistic nature of flood flows and is computed either from a statistical analysis of gaged streamflow data or through rainfall-runoff modeling. Stage versus discharge relationships are developed from water-surface profile computations. A stage at an index location corresponds to a water-surface profile along the river reach. The stage versus damage relationship represents the damage, in dollars, which would occur along a river reach if floodwaters reach various levels. Alternative approaches for developing stage versus damage relationships involve using: historical flood damage data for the study area; synthetic data for the study area; or generalized local, regional, or national inundation depth versus percent damage functions applied to an inventory of property located in the floodplain.

213. A river system is divided into reaches for analysis purposes. Average annual damages are computed for each reach and summed to get the total. Each reach is represented by an index location. The functional relationships are developed for each index location and represent the variables for the entire reach.

214. In order to model the effects of reservoir storage capacity, a series of flood hydrographs representing a broad range of magnitudes must be routed through the stream system. Each flood provides one point on the basic relationships. Each flood consists of a set of inflow hydrographs to the stream system. Hydrographs are included on each tributary at a location upstream of all damage areas and damage reduction measures. Additional hydrographs are included at downstream locations to reflect incremental lateral inflows.

Early simulation models

215. Simulation modeling of major river basins began in the United States in 1953 with a study by the Corps of Engineers of the operation of six reservoirs on the Missouri River (Manzer and Barnett 1966). The

objective was to maximize power generation subject to constraints imposed by specified requirements for navigation, flood control, and irrigation. The International Boundary and Water Commission simulated a two-reservoir system on the Rio Grande River in 1954. A simulation study for the Nile River Basin in Egypt in 1955 entailed alternative plans with as many as 17 reservoirs of hydropower sites. The objective was to determine the particular combination of reservoirs and operating procedures which would maximize the volume of useful irrigation water (Manzer and Barnett 1966). Pioneering research in developing reservoir system simulation methods was accomplished in conjunction with the Harvard Water Program (Maass et al. 1966). Hufschmidt and Fiering (1966) discuss the simulation modeling work of the Harvard Water Program and application to the multipurpose planning in the Lehigh River Basin.

State-of-the-art simulation models

216. Several major reservoir system simulation models considered to be representative of the current state of the art are cited below. These particular models are addressed because they are illustrative of simulation modeling in general, and they have been extensively used and/or frequently referenced in the literature.

217. HEC-5. In the sense of being applicable to a wide range of reservoir operation problems, the HEC-5 Simulation of Flood Control and Conservation Systems computer program (HEC 1982a) is probably the most versatile of the available models. It is also totally generalized for application to any reservoir system as opposed to other models which were developed for a specific river basin. HEC-5 is well documented and has been used in a relatively large number of studies. An initial version released in 1973 has subsequently been significantly expanded several times leading up to the 1985 version. The user's manual (HEC 1982a) provides detailed instructions for using the generalized computer program. Feldman (1981) describes HEC-5 as well as the several other water resources system simulation models available from the Hydrologic Engineering Center (HEC) of the USACE. The HEC reports (1976, 1977b) discuss simulation modeling in general but with particular reference to HEC-5. Papers by Eichert, Peters, and Pabst (1975), McMahon, Bonner, and Eichert (1979), and Eichert (1979) describe specific applications of the model. Sullivan (1989) applied HEC-5 to a case study investigation of analyzing reservoir operations from a military perspective.

218. HEC-5 simulates the operation of multipurpose, multireservoir systems. The reservoir system consists of a number of reservoirs and control

points. Water demands for municipal, industrial, and/or agricultural water use, hydropower, or instream flow maintenance are specified at the reservoirs or at downstream control points. Flood control storage is operated based on flows at downstream control points. The model operates the system of reservoirs in order to best meet specified flood control and conservation requirements.

219. HEC-5 may be used to determine reservoir storage requirements and/or operational strategies for various water control needs. The model is also used to assist in determining reservoir releases during real-time flood control operations. Capabilities are provided for computing expected annual flood damages and hydropower benefits. A program option is also provided to determine the firm yield for either water supply or hydroelectric power.

220. Since the program has no rainfall-runoff modeling capability, streamflows must be furnished as input data. The simulation may be performed using any one-hour or larger time interval. The time interval may vary during a simulation. For example, conservation operation is typically modeled with monthly flows, switching to daily or hourly flows for modeling operations during flood events. Several hydrologic routing methods are provided, including modified Puls, Muskingum, average lag, and working R&D. The reservoir rule curve can vary monthly. Storage in each reservoir is discretized into levels or pools for operational control purposes. The model uses a set of operational priorities for dealing with conflicts between multiple-purpose objectives and to balance storage between reservoirs.

221. HEC-3. The HEC-3 Reservoir System Analysis for Conservation computer program is similar to HEC-5 in regard to conservation, except the HEC-3 hydropower capabilities are not as extensive as HEC-5. HEC-3 has no flood control simulation capability. Input and output are virtually the same as HEC-5 for their common capabilities (HEC 1981).

222. SWD model. A generalized reservoir regulation model developed by the Southwestern Division (SWD) of the USACE is described by Hula (1981). Application of the model to the Arkansas River System is described by Coomes (1981) and Copley (1981). The division and district offices in the five-state SWD have routinely applied the model for a number of years. The SWD model simulates the daily sequential regulation of a multipurpose reservoir system. The model performs the same types of hydrologic and economic simulation computations as HEC-5. The SWD model uses a one-day computation interval whereas

HEC-5 uses a variable time interval. Details of handling input data and various computational capabilities differ somewhat between the two models.

223. Hydrologic input data include daily uncontrolled streamflows at each reservoir and river control point and daily evaporation at each reservoir. Economic input data include: stage-damage curves; stage-discharge curves; stage-area curves; cropping patterns; crop values; navigation costs relative to discharge; dredging requirements relative to discharge and duration; recreation benefits as a function of pool elevation, season, and pool fluctuation; hydroelectric power value; and costs for purchasing thermal electric power as a function of season and time of day. Input data describing the physical characteristics of the reservoir-stream system include: reservoir elevation-area-capacity curves; reservoir discharge capacity; hydroelectric power plant capacity; tailwater rating curves; and Muskingum routing coefficients. Reservoir release requirements and constraints are based on controls at the reservoir and downstream control points. Hydrologic information provided by the model includes: monthly and annual frequency plots of maximum and minimum reservoir storage and control point discharge; duration plots of reservoir pool elevation and control point discharge; and water supply and low flow shortages. Economic output includes flood damages, recreation benefits, power value, cost of purchased power, dredging costs, and navigation costs.

224. SSARR model. The Streamflow Synthesis and Reservoir Regulation (SSARR) model was developed by the North Pacific Division (NPD) of the USACE primarily for streamflow and flood forecasting and reservoir design and operation studies. Various versions of the model date back to 1956. A program description and user manual (NPD 1975) documents the present version of the computer program. Numerous reservoir systems, including the Columbia River Basin in the United States and Mekong River Basin in Southeast Asia, have been modeled with the generalized computer program by various agencies, universities, and other organizations.

225. The SSARR computer program simulates the hydrology of a river system. The model is comprised of three basic components: (a) a watershed model for synthesizing runoff from rainfall and snowmelt, (b) a streamflow routing model, and (c) a reservoir regulation model for analyzing reservoir storage and outflow.

226. TWDB models. The Texas Water Development Board (TWDB) began development of a series of surface-water simulation models in the late 1960's in conjunction with formulation of the Texas Water Plan (TWDB 1974). The

present RESOP-II, SIMYLD-II, AL-V, and SIM-V computer programs evolved from earlier versions.

227. The Reservoir Operating and Quality Routing Program (RESOP-II) is designed for performing a detailed analysis of the annual yield of a single reservoir. A quality routing option adds the capability to route up to three nondegradable constituents through a reservoir and to print a frequency distribution table and a concentration duration plot for the calculated end-of-month quality of the reservoir (Browder 1978).

228. SIMYLD-II provides the capability for analyzing water storage and water transfer within a multireservoir or multibasin system (TWDB 1974). SIMYLD-II simulates the operation of a system subject to a specified sequence of demand and hydrologic conditions. The model simulates catchment, storage, and transfer of water within a system of reservoirs, rivers, and conduits on a monthly basis with the object of meeting a set of specified demands in a given order of priority. If a shortage occurs such that not all demands can be met for a particular time period, the shortage is located at the lowest priority demand node. SIMYLD-II also provides the capability to determine the firm yield of a single reservoir within a multireservoir water resources system. An iterative procedure is used to adjust the demands at each reservoir of a multireservoir system in order to converge on its maximum firm yield at a given storage capacity assuming total system operation. While SIMYLD-II is capable of analyzing multireservoir systems, it is not capable of analyzing a single reservoir as accurately as RESOP-II. Consequently, SIMYLD-II and RESOP-II are both used in an interactive manner to analyze complex systems.

229. The Surface Water Resources Allocation Model (AL-V) and Multireservoir Simulation and Optimization Model (SIM-V) simulate and optimize the operation of an interconnected system of reservoirs, hydroelectric power plants, pump canals, pipelines, and river reaches (Martin 1981, 1982, 1983). SIM-V is used to analyze short-term reservoir operations. AL-V is for long-term operations. The models combine simulation and optimization. The steady-state operation of a surface water system is represented as a network flow problem. The out-of-kilter linear programming algorithm is used to analyze capacitated networks. Hydroelectric benefits are incorporated by solving successive minimum-cost network flow problems, where flow bounds and unit costs are modified between successive iterations to reflect first-order changes in hydroelectric power generation with flow release rates and reservoir storage.

230. PRISM. The Department of Geography and Environmental Engineering at John Hopkins University performed a study sponsored by the Office of Water Research and Technology on the operation of reservoirs in the Potomac River Basin and water supply management in the Washington, DC, Metropolitan Area (Palmer et al. 1980, 1982). The first year of the study focused on the formulation and solution of optimization models, and the second year focused on development of the Potomac River Interactive Simulation Model (PRISM). PRISM provided a much more detailed representation of the water supply system than the optimization models.

231. PRISM simulates the operation of the four reservoirs and the allocation of water within the Washington Metropolitan Area. Input data include: (a) weekly streamflow into each reservoir and weekly flow of the Potomac River, (b) weekly water use demand coefficients for each of three water supply agencies, (c) an allocation formula for distribution of water to jurisdictions, and (d) rules and constraints for operating the reservoirs in the system. The model determines on a weekly basis the supply of water available to each of the three jurisdictions resulting from previous decisions made in response to information on the state of the system.

232. PRISM is designed for use in a batch mode or in an interactive mode. When operating in the batch mode, decision strategies are specified by the user prior to model execution, and PRISM performs the function of the regional water supply manager in strict accordance with rules provided by the model user. The interactive mode allows participants to engage in a dialogue with the model as it is being executed, thereby changing model parameters and overriding prespecified decision rules. The interactive model represents an attempt to include, in a formal analytical modeling exercise, the process by which water supply management decisions are made.

233. MIT simulation model. Strzepek and Lenton (1978) describe the Massachusetts Institute of Technology (MIT) River Basin Simulation Model and its application to the Vardar/Axios Basin in Yugoslavia and Greece. A user's manual is provided by Strzepek et al. (1979). The generalized computer program provides the capability to evaluate the hydrologic and economic performance of a river basin development system. Existing and proposed reservoirs, hydroelectric power plants, thermal power stations, irrigation areas, and diversions and withdrawals for municipal, industrial, and other uses are represented in the model as a system of arcs and nodes. The model computes the monthly flows at all nodes in the basin, given the streamflows at the

start nodes. System reliability in meeting water demands is assessed. Irrigation, hydroelectric power, and municipal and industrial water supply benefits are computed and compared with project costs.

234. Trent River System model. Sigvaldason (1976) describes a simulation model developed to assess alternative operation policies for the 48-reservoir multipurpose water supply, hydropower, and flood control system in the Trent River Basin in Ontario, Canada. The model was originally developed for planning but has also been used for real-time operation. In the model, each reservoir was subdivided into five storage zones. Time-based rule curves were prescribed to represent ideal reservoir operation. The combined rule curve and storage zone representation is similar to that in HEC-5. Ranges were prescribed for channel flows, which were dependent on water-based needs. Penalty coefficients were assigned to those variables which represented deviations from ideal conditions. Different operational policies were simulated by altering relative values of these coefficients. The development and use of the model were simplified by representing the entire reservoir system in capacitated network form and deriving optimum solutions for individual time periods with an out-of-kilter algorithm. Except for differences in the objective functions, the optimization submodel for achieving optimal responses during individual time intervals is similar to the approach used in the Texas Water Development Board models.

## Optimization Models

235. During World War II, the Allies organized interdisciplinary teams to solve complex scheduling and allocation problems involved in military operations. Mathematical optimization models were found to be very useful in this work. After the war, the evolving discipline of operations research or management science continued to rely heavily upon optimization models for solving a broad range of problems in private industry. The same mathematical programming techniques also became important tools in the various systems engineering disciplines, including water resources systems engineering. Reservoir operations have been viewed as an area of water resources planning and management with a particularly high potential for beneficial application of optimization models.

236. The literature related to optimization models in general and application to reservoir operation in particular is extensive. The various

optimization techniques are treated in depth by numerous mathematics, operations research, and systems engineering textbooks. Application of optimization techniques to reservoir operation problems has been a major focus of water resources planning and management research during the past two decades. The textbook by Loucks, Stedinger, and Haith (1981) explains the fundamentals of applying optimization techniques in the analysis of water resources systems. Yeh (1982) reviews the state of the art of optimization models applied to the operation of reservoir systems. Wurbs et al. (1985) provide a state-of-the-art review and annotated bibliography of systems analysis techniques applied to reservoir operation, which is directed toward optimization, simulation, and stochastic analysis methods. A majority of the over 700 references cited in their bibliography focus on optimization techniques.

237. There is no generalized model for optimizing reservoir operations. Rather, optimization models have been formulated for a variety of specific types of reservoir operation problems. The models have usually been developed for a specific reservoir system. University research projects involving case studies account for most of the applications of optimization techniques to reservoir operations to date. Major reservoir systems for which optimization models have been used to support actual operations decisions include the California Central Valley Project and Tennessee Valley Authority System (Yeh 1982).

238. Most of the applications of optimization techniques in reservoir systems analysis involve linear programming, dynamic programming, or combining a simulation model with a search algorithm. The numerous other available nonlinear programming techniques have been used relatively little in reservoir planning and operation.

239. Optimization models are formulated in terms of determining values for a set of decision variables which will maximize or minimize an objective function subject to constraints. The objective function and constraints are represented by mathematical expressions as a function of the decision variables. For a reservoir operation problem, the decision variables might be release rates or end-of-period storage volumes. The objective function to be maximized could be the quantitative measure of economic benefits for various project purposes, hydroelectric energy produced, firm yield, a water quality index, or the length of the navigation season. Likewise, an objective function to be minimized could be expressed as deviations from target discharges, a shortage index such as the squared sum of deviations between target and

actual discharges, the volume of water released to meet minimum flow requirements, economic costs due to water shortages, expected annual flood damages, or any number of other indices of system performance. Constraints typically include physical characteristics of the reservoir-stream system and minimum diversion or low flow requirements for various purposes.

Linear programming

240. Linear programming is the simplest and most widely used of the optimization techniques. Its popularity in water resources systems analysis, as well as in operations research and other systems engineering disciplines, is due largely to the following factors:

a. The technique is applicable to a wide variety of problems.

b. An efficient solution algorithm is available.

c. Numerous generalized computer packages are readily available for solving linear programming problems.

241. Linear programming consists of minimizing or maximizing a linear objective function subject to a set of linear constraints, expressed as follows:

$$\begin{array}{ll} \text{maximize (or minimize)} & Z = \sum c_j x_j \\ \text{subject to} & a_{ij} x_j \leq b1 \\ & \text{for } i = 1,2,\ldots,m \\ \text{and} & x_j \geq 0 \\ & \text{for } j = 1,2,\ldots,n \end{array}$$

where Z is the objective function; $x_j$ represents the decision variables; $c_j$, $a_{ij}$, and $b_i$ are constants; n is the number of decision variables; and m is the number of constraints. The "less than equal" sign in the constraint inequalities may be replaced by "greater than or equal" or "equal" signs to suit the particular problem being modeled.

242. If a problem can be properly formulated in the required mathematical format, solving for the optimum values for the decision variables is straightforward. The simplex algorithm, used in essentially all linear programming computer packages, is explained in detail in a number of textbooks, including Wagner (1975). Under certain assumptions, nonlinear problems

can be linearized and solved by iteration or approximation procedures. Yeh (1982) and Wurbs et al. (1985) review extensions of linear programming applied to reservoir operations, which have been reported in the literature, including the linear decision rule, chance contraints, and stochastic programming with recourse.

Dynamic programming

243. Dynamic programming is not a precise algorithm like linear programming, but rather a general approach to solving optimization problems. Application of dynamic programming depends on the ingenuity of the modeler, with particular equations used in the model being developed for the specific problem. The dynamic programming approach involves decomposing a complex problem into a series of simpler subproblems which are solved sequentially, while transmitting essential information from one stage of the computations to the next using state concepts.

244. Dynamic programming models have the following characteristics:

a. The problem is divided into stages with a decision required at each stage. The stages may represent different points in time (as in determining reservoir releases for each time interval), different points in space (for example, releases from different reservoirs), or different activities (such as releases for different project purposes or water uses).

b. Each stage of the problem must have a finite number of states associated with it. The states describe the possible conditions in which the system might be at that stage of the problem. The amount of water in storage is an example of a possible state variable.

c. The effect of a decision at each state of the problem is to transform the current state of the system into a state associated with the next stage. If the decision is how much water to release from the reservoir at the current time, this decision will transform the amount of water stored in the reservoir from the current amount to a new amount for the next stage.

d. A return function indicating the utility or effectiveness of the transformation is associated with each potential state transformation. The return function allows the objective function to be represented by stages.

e. The optimality of the decision required at the current stage is judged in terms of its impact on the return function for the current stage and all subsequent stages.

245. The fundamentals of dynamic programming are outlined by Wagner (1975) and Loucks, Stedinger, and Haith (1981) as well as other textbooks. Yeh (1982) and Wurbs et al. (1985) review extensions to dynamic programming

which have been applied to reservoir operation problems, including multiple objective, differential, reliability constrained, and stochastic dynamic programming.

246. Linear programming, when compared to dynamic programming, is better defined, easier to understand, and readily available in the form of generalized computer programs. Many reservoir operation problems can be represented realistically by a linear objective function and set of linear constraints. Various linearization techniques have been used successfully to deal with nonlinearities. However, the strict linear form of the mathematical model does significantly limit its applicability. Dynamic programming is applicable to reservoir operation problems which can be formulated as optimizing a multistage decision process. Nonlinear properties of a problem can be readily reflected in a dynamic programming formulation. However, various assumptions, including a separable objective function, limit the range of applicability and require ingenuity and understanding by the modeler in applying dynamic programming. The so-called "curse of dimensionality" is a major consideration in dynamic programming. Increasing the number of state variables greatly increases the computational burden.

Search techniques

247. In certain reservoir systems analysis situations, values of a defined objective function can be computed using a simulation model even though system complexities preclude the use of mathematical programming techniques. The simulation model could be run repeatedly in a trial-and-error search for an optimum decision policy. An alternative approach is to combine the simulation model with a search algorithm. The search algorithm adjusts values of the decision variables in a systematic iterative fashion while exercising the simulation model each time a value is needed for the objective function. The overall model incorporating the search algorithm with the simulation model automatically computes values of the decision variables which optimize the objective function. The flood damage-reduction-system optimization option in HEC-1 is an example of this approach (HEC 1985).

## Comparison of Simulation and Optimization Models

248. From the perspective of beir widely accepted and applied by the reservoir development and management community, the state of the art of reservoir systems analysis is simulation. Simulation models have been routinely

applied for many years by the water agencies and other entities responsible for planning, design, and operation of reservoir projects. Simulation models typically provide an accounting system for tracking the volumes of water in storage and flows at pertinent locations in a stream-reservoir system over time for given hydrologic inputs and operating policies. Simulation models also provide a framework for estimating economic benefits and costs for a given system configuration and operating policy based on given economic input data. Optimization strategies often consist of numerous systematic trial-and-error runs of a simulation model with alternative decision policies.

249. The academic research community in particular, and many practitioners as well, have been extremely enthusiastic about optimization, in the sense of mathematical programming techniques, applied to reservoir operation problems. During the past 20 years, a major thrust of research and the resulting literature related to reservoir operation has been to supplement simulation models with mathematical programming techniques. The characteristics of certain reservoir operation problems are ideally suited for applying linear and dynamic programming and various other nonlinear optimization algorithms. Research results, case studies, and experience in application of optimization models in actual planning and real-time operation decisions indicate a high potential for improving reservoir operations through their use. However, optimization techniques have played a relatively minor role compared to simulation models in regard to influencing decisions made in actual project planning and operation.

250. Mathematical programming or optimization models automatically compute values for the decision variables which optimize an objective function. Since simulation models are limited to predicting the system performance for a given decision policy, optimization models have a distinct advantage in this regard. However, simulation models have certain other advantages over optimization models. Optimization typically requires significant simplifications in the mathematical representation of a system. Simulation models generally permit more detailed and realistic representation of the complex hydrologic and economic characteristics of a reservoir system. Stochastic analysis methods can be combined with simulation models easier than with optimization models. The concepts inherent in simulation tend to be easier to understand and communicate than optimization modeling concepts.

251. Optimum sizing of storage capacities, establishment of release policies, real-time operations, and evaluating impacts of various actions are

complex tasks involving numerous hydrologic, economic, environmental, institutional, and political considerations. Defining system objectives, developing criteria for quantitatively measuring system performance in fulfilling the objectives, and handling interactions and conflicts between objectives comprise a major area of complexity. Mathematical optimization techniques require that the real system be represented in the proper mathematical format. Representing complex project objectives and performance criteria in the required format, without unrealistic simplifications, is a particularly difficult aspect of the modeling process which limits the application of optimization techniques.

252. Simulation, with either deterministic or synthetically generated stochastic hydrologic inputs, will likely continue to be the "work-horse" of reservoir system analysis. The more mathematically sophisticated optimization methods will provide valuable supplemental analysis capabilities for a select number of specific types of problems.

253. Optimization and simulation can also be used in combination. Preliminary screening with an optimization model may be used to develop a manageable range of alternative decision policies for further detailed analysis with a simulation model. The Potomac River Study (Palmer et al. 1980) is an example of this general approach. Another strategy, illustrated by the Tennessee Valley Authority HYDROSIM model (Gilbert and Shane 1982), is for an optimization model to be embedded as a component of a complex simulation model. Likewise, an optimization model may search for an optimum decision policy while activating a simulation model to compute the objective function value for any given set of decision variable values. Ford, Garland, and Sullivan (1981) provide an example of combining a reservoir operation simulation model with a nonlinear optimization algorithm.

## Flood Wave Analysis

254. Unsteady flow conditions in a river-reservoir system are caused by precipitation runoff and/or reservoir releases. Flood wave analysis (or unsteady flow modeling) consists of mathematically predicting the changing flow characteristic (such as discharge, velocity, depth, and celerity) as a function of time and location.

## Applications

255. Flood wave analysis plays an important role in a variety of reservoir modeling applications. For example, hydroelectric power operations often involve rapid changes in release rates. Unsteady flow-modeling techniques are used to analyze the characteristics of the resulting surge in order to predict impacts on downstream properties. Travel time is an important consideration in water supply operations in situations where water is diverted from the river many miles below the dam making the release.

256. Simulating flood control operations involves computing release rates for each consecutive time interval which will empty the reservoir as quickly as possible without exceeding allowable flow rates at downstream control points. Reservoir releases are routed to the downstream control points and combined with incremental lateral inflows. An iterative procedure is required to determine the release rate for which the routed hydrograph results in the correct discharges at the control points. Thus, the routing computations may be repeated a number of times for each reservoir release rate determination.

257. Other applications involve predicting the effects of a major flood event on a stream-reservoir system. Flood insurance studies and floodplain zoning may require delineation of the 100-year return interval floodplain downstream and/or upstream of a dam. Dam safety studies require modeling the passage of extreme flood events through a reservoir to evaluate spillway adequacy and modeling the downstream effect of a flood wave caused by postulated dam breach.

258. Analyzing induced flood waves caused by gate releases or a breached dam can be expected to be a major thrust of military applications of reservoir modeling. Flood wave analysis techniques can be used in predicting damaging impacts on military and civilian facilities and activities. Flow characteristics at downstream crossing sites associated with various reservoir release conditions may be a major concern in predicting trafficability.

### Flood wave analysis methods

259. Military Hydrology Reports 9 and 13 (Wurbs 1985, 1986) provide a review of the state of the art, including an extensive bibliography, of flood wave modeling. Although developed from the perspective of dam-breach flood forecasting, these reports are pertinent to flood wave modeling in general. The present discussion is an overview summary of flood wave analysis methods, but from the perspective of modeling reservoir operations.

260. Flood wave analysis methods can be categorized as outlined in Table 6. Although a flood wave is a three-dimensional phenomenon, most modeling is based on the assumption of one-dimensional flow. Two general approaches can be taken in one-dimensional modeling of unsteady flows in rivers and reservoirs: (a) hydraulic routing or (b) hydrologic routing methods used in combination with steady flow water-surface profile computations. Hydraulic routing involves simultaneously computing discharges, velocities, and stages as a function of time and location. The alternative approach is to compute discharge hydrographs using hydrologic routing and then relate stage to discharge as a separate computation using steady, uniform, or gradually varied flow techniques.

261. A broad range of applications and modeling complexities have resulted in the development over the years of numerous flood wave analysis techniques. Several alternative methods are listed in Table 6 for each of the categories of hydraulic routing, hydrologic routing, and steady flow water-surface profile computations.

Table 6

Flood Wave Analysis Methods

I. Two- and three-dimensional models

II. One-dimensional models
   - A. Dynamic routing
   - B. Generalized dimensionless relationships
   - C. Simplified hydraulic routing methods
     1. Kinematic and diffusion wave models
     2. Linearization of the St. Venant equations
   - D. Hydrologic storage routing
     1. Modified Puls and variations
     2. Muskingum and variations
     3. Variable storage coefficient
     4. Modified attenuation-kinematic routing
   - E. Purely empirical methods

262. Hydraulic routing methods are based on the St. Venant equations, which are two partial differential equations expressing the physical laws of conservation of mass and conservation of momentum. Dynamic routing involves solution of the complete St. Venant equations. Simplified hydraulic-routing techniques have been developed based on neglecting certain terms or otherwise simplifying the St. Venant equations. Another approach for providing simplified routing capabilities has been to precompute generalized dimensionless curves using a dynamic routing model. Most hydrologic routing methods are based upon the storage form of the continuity equation, which is an alternative way of expressing the concept of conservation of mass, and a relationship between storage and either outflow and/or inflow. Other hydrologic routing methods are purely empirical and based strictly on intuition and observation of past floods. Flow depths for a given discharge can be computed based on the simplifying assumption of either uniform flow or gradually varied flow.

263. Two- and three-dimensional models. The flow characteristics of a flood wave actually vary in three dimensions. Floodplain irregularities such as abrupt contractions and expansions in valley topography, tributaries, bridges, control structures, and overtopped levees cause accelerations with horizontal and vertical components perpendicular to the flow axis. Water may flow laterally outward from the river channel to fill overbank floodplain storage as the stage rises and then laterally back toward the channel as the stage falls. Three-dimensional accelerations can be expected to be particularly significant near the outlet structures through which water is released from a reservoir.

264. Unsteady flow equations can be expressed in various forms which reflect components of flow in the three directions of a Cartesian coordinate system. Two-dimensional flow modeling is much more common than three-dimensional modeling. The few fully three-dimensional models that now exist are in a developmental stage. In two-dimensional modeling, acceleration in the vertical direction is typically assumed to be negligible so that the equations can be written for flow components in the two horizontal directions. The state of the art of two-dimensional flow modeling has advanced considerably during the past decade caused in part by advances in computer technology. However, this progress has been primarily associated with coastal and estuarine applications. Application of two-dimensional methods to flood routing in rivers and reservoirs has been much more limited.

265. St. Venant equations. Hydraulic routing is based on the one-dimensional unsteady flow equations derived by Barre De Saint Venant in 1871. The St. Venant equations consist of a conservation of mass (continuity) equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial Q}{\partial t} + q = 0$$

and a conservation of momentum (dynamic) equation:

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial h}{\partial x} + gS_f + \frac{Vq}{A} = 0$$

where

- $Q$ = discharge
- $x$ = distance along the waterway
- $t$ = time
- $q$ = lateral outflow (lateral inflow negative) per unit length of waterways
- $V$ = mean velocity = $Q/A$
- $g$ = gravitational acceleration constant
- $h$ = water-surface elevation above a datum
- $S_f$ = friction slope
- $A$ = cross-sectional area

266. A number of references, including Chow (1959) and Cunge, Holley, and Verway (1980), contain derivations of the one-dimensional unsteady flow equations. The conservation of mass equation expresses the fundamental principle that inflow minus outflow equals change in storage over time. The conservation of momentum equation is derived based on Newton's second law of motion, which states that the sum of all the external forces acting on a system of particles is equal to the time rate of change of linear momentum of the system. The terms in the conservation of momentum equation are expressions of local acceleration $\frac{\partial V}{\partial t}$, convective acceleration $V \frac{\partial V}{\partial t}$, pressure and gravity forces $g \frac{\partial h}{\partial x}$, frictional forces ($g\ S_f$), and acceleration of lateral inflow $\frac{Vq}{A}$. The last term is usually omitted, assuming the momentum of lateral inflows to be negligible. The friction slope ($S_f$) is estimated using a steady flow empirical uniform flow formula such as the Manning equation.

267. Dynamic routing. A flood wave analysis model based on the complete St. Venant equations is called a dynamic wave (or dynamic routing) model. Dynamic routing is the most theoretically correct of the numerous one-dimensional flood-routing methods. Dynamic routing is the only method which accounts for the acceleration effects of a flood wave and backwater effects caused by dams, channel constrictions, tributary inflows, and bridges. Improvements in accuracy provided by dynamic routing over simpler hydraulic and hydrologic methods are most pronounced in situations where wave acceleration effects are significant compared to the effects of gravity and channel friction. Consequently, dynamic routing is particularly pertinent to modeling reservoir releases from rapid gate openings and dam-breach floods as compared to more gradually varied precipitation floods. Dynamic routing is also particularly advantageous for reaches upstream of dams and for flat channel slopes where backwater effects are pronounced.

268. The St. Venant equations have no known general analytical solution. The equations have been solved analytically only under very restrictive and simplifying conditions, with the solutions generally not acceptable for practical problems. Due to the mathematical complexity of the theoretical equations, for many years significant simplifications were necessary in order to obtain solutions. However, during the last two decades, solution of the complete St. Venant equations has become practical using numerical methods and high-speed computers.

269. Numerical solution techniques convert the differential equations into algebraic difference equations that may be solved for Q (or V) and h (or y) at finite incremental values of x and t, given initial and boundary conditions. Various numerical techniques can be used, but all involve replacing the derivatives in the equations by finite difference approximations of some form. The differential equations are approximated with finite difference algebraic equations which are solved through a series of algebraic operations proceeding stepwise through time and distance.

270. Generalized dimensionless relationships. Although significant advances have been made in recent years in the development of practical generalized dynamic wave computer programs, significant expertise, time, and computer resources are required to apply the models. One strategy for providing the capability for an expedient analysis is to develop precomputed dimensionless relationships. The generalized routing model consists of a family of dimensionless curves which have been developed using a dynamic routing model.

The Simplified Dam-Breach Flood Forecasting Model developed by the National Weather Service (Wetmore and Fread 1981) and the Dimensionless Graph Procedure developed by the Hydrologic Engineering Center (Sakkas 1980) are based on the dimensionless relationship approach. Both of these flood-routing methods were developed specifically for analyzing dam-breach floods. Significant simplifications were required to develop a set of dimensionless curves which can be applied to a wide range of situations using a few parameters. The assumption of a prismatic channel of specified shape is one of the major simplifications made.

271. Kinematic and diffusion wave models. Various simplifications to the St. Venant equations have been made to overcome difficulties in obtaining solutions. Kinematic and diffusion wave models are based on the St. Venant equation for conservation of mass and a simplified form of the conservation of momentum equation in which certain terms are neglected.

272. In kinematic routing, the momentum of the unsteady flow is assumed to be the same as that of steady uniform flow. The friction slope ($S_f$) is equal to normal slope $S_o = \partial y/\partial x - \partial h/dx$, where h is water-surface elevation above a datum and y is flow depth. Thus, the conservation of momentum equation is simplified to $S_f - S_o = 0$. Discharge is a single-valued function of stage which can be expressed by a uniform flow formula such as the Manning equation in the form $A = \alpha Q^{\beta}$. Combining these expressions with the conservation of mass equation results in the following kinematic wave equation:

$$\frac{\partial Q}{\partial x} + \alpha \beta Q^{\beta-1} \frac{\partial Q}{\partial t} = 0$$

which can be solved analytically or by various finite difference techniques.

273. The kinematic wave model reflects only translation of the flood wave and the attendant distortion that is attributed to kinematic effects of wave movement. Errors inherent in finite difference solutions cause an attenuation and dispersion of the flood wave. However, this numerical attenuation merely mimics the actual physical phenomenon. The kinematic wave equation contains no mechanism to model attenuation. Also, downstream backwater effects are not reflected in the model.

274. The diffusion model is based on the St. Venant conservation of mass equation and the following simplified form of the momentum equation:

$$S_f - \partial h/\partial X = 0$$

Inclusion of the water slope term ($\partial h/\partial X$) provides a significant improvement over the kinematic model. This term allows the diffusion model to reflect the attenuation effect of the flood wave. It also allows the specification of a boundary condition at the downstream end of the routing reach to account for backwater effects. The diffusion model does not include the inertia terms, $\frac{\partial V}{\partial t}$ and $V \frac{\partial V}{\partial X}$, of the St. Venant momentum equation. Therefore, diffusion routing is limited to relatively gradually rising flood waves in channels of fairly uniform geometry.

275. Hydrologic storage routing. Hydrologic routing is based on the conservation of mass or continuity equation

$$I - O = dS/dt$$

where I is inflow rate, O is outflow rate, and dS/dt is rate of change in storage with respect to time. The continuity equation can be written in the following finite difference form:

$$(I_1 + I_2)/2 - (O_1 + O_2)/2 = (S_2 - S_1)/\Delta t$$

where subscripts 1 and 2 refer to the beginning and end, respectively, of the routing interval $\Delta t$. A relationship between storage and discharge must be combined with the continuity equation to obtain a routing model. All hydrologic storage routing methods use the continuity equation. Variations between methods are due to the different ways of expressing the storage versus discharge relationship.

276. Modified Puls routing. The modified Puls or storage indication method and its variations are based on the assumption that storage is a single-valued function of outflow (Linsley, Kohler, and Paulhus 1982). The continuity equation can be written as

$$2S_2/\Delta t + O_2 = I_1 + I_2 + 2S_1/\Delta t - O_1$$

which can be solved step-by-step for the left-hand side, with the right-hand side of the equation known at each step of the computations. A relationship between the left-hand side of the equation and outflow must be developed from a known storage-outflow relationship. A reservoir storage-outflow relationship is developed from a reservoir elevation-storage relationship and rating curves for the outlet structures. For a gated reservoir, a family of storage-outflow relations must be developed for a range of gate openings.

277. The modified Puls methods, or variations thereof, is widely used for routing flood hydrographs through reservoirs. The assumption that storage is a unique function of outflow is valid for a reservoir that is short and deep enough for the water surface to be horizontal.

278. Although most applicable for reservoirs, the method is also used for channel routing. Storage-discharge relationships for a channel reach can be approximated by assuming uniform flow and using a uniform flow formula such as the Manning equation with a cross-section representative of the reach. A somewhat better storage-discharge approximation can be achieved using water-surface profiles computed assuming steady gradually varied flow. The input data for both of these approaches are cross sections and roughness coefficients. A third approach is to develop a storage-discharge function for a reach from historical inflow and outflow hydrographs.

279. Using modified Puls routing for channels is complicated by the fact that the degree of flood wave attenuation varies with the reach length used in the computations. A routing reach is typically divided into subreaches for computational purposes. The number of subreaches can be used as a calibration parameter but is difficult to precisely estimate if historical flow data are not available for calibration.

280. Muskingum routing. The Muskingum method and its variations are based on the assumption of a linear relationship between storage and a weighted combination of inflow and outflow

$$S = K\ (xI + (1 - x)O)$$

where K is a storage time constant and x is a weighting factor which varies between 0 and 0.5 for a given reach (Linsley, Kohler, and Paulhus 1982). Combining this expression with the conservation of mass equation results in the Muskingum routing equation

$$O_2 = c_o I_2 + c_1 I_1 + c_2 O_1$$

where

$c_o = -(Kx + 0.5\ \Delta t)/(K - Kx + 0.5\ \Delta t)$

$c_1 = (Kx + 0.5\Delta t)/(K - Kx + 0.5\Delta t)$

$c_2 = (K - Kx - 0.5\Delta t)/(K - Kx + 0.5\Delta t)$

Values for the parameter K can be approximated as the travel time through the reach, and an average value of 0.2 can be assumed for x. Better values of K and x can be determined from observed inflow and outflow hydrographs for the reach of river.

281. A modified version of the Muskingum method developed by Cunge (1969) assumed a single-value depth-discharge relationship, which is equivalent to a kinematic wave model, and applied a four-point finite difference technique to derive expressions for the routing coefficients x and K. After the constants x and K are determined for a particular river reach, the routing computations are identical to the original Muskingum method. The Muskingum-Cunge method does not require observed inflow and outflow hydrographs to establish the routing coefficients as required in the Muskingum method, but best results are obtained if some actual flow data are available.

282. Numerous other variations of the Muskingum method have been developed including the Kalinin-Miljukov, SSARR, and lag and route methods (Fread 1982). Muskingum-type models provide best results when applied to slowly fluctuating rivers with negligible lateral inflows and backwater effects.

283. Working R and D routing. The working R and D method combines concepts of both modified Puls and Muskingum routing (USACE 1960). The method is based on the assumption that

$$D = xI + (1 - x)\ O$$

where the "working" discharge (D) represents a hypothetical steady flow that would result in storage, in a given reach, equal to that produced with given values of actual inflow (I) and outflow (O). The variable x is a weighting factor. This equation is rearranged to obtain the routing equation

$$O_2 = D_2 - (x / (1 - x)) (I_2 - D_2)$$

where subscript refers to the end of the routing interval. The working R, or storage indication, is given by

$$R = S/\Delta t + D/2$$

At each time step, R is computed from the equation

$$R_2 = R_1 + (I_1 + I_2)/2 - D_1$$

$D_2$ is interpolated from D versus R data, and $O_2$ is computed from the routing equation given above.

284. Variable storage coefficient routing. The variable storage coefficient routing method was developed by Williams (1969). The following expression for travel time (T)

$$T = L/V = S/Q$$

where

L = reach length

V = average velocity

S = storage in reach

Q = average discharge

is combined with the conservation of mass equation to obtain the following variable storage coefficient routing equation:

$$O_2 = c_2 I_a + (1/c_1 - 1)\, O_1$$

$$c_1 = 2\Delta t \,/\, (2T_1 + \Delta t)$$

$$c_2 = 2\Delta t \,/\, (2T_2 + \Delta t)$$

$$I_a = (I_1 + I_2) \,/\, 2$$

where subscripts 1 and 2 refer to the beginning and end of the routing interval $\Delta t$. Travel time is computed for an average of inflow and outlet velocities along with a correction for variation in water-surface slope.

285. Thus, the variable storage coefficient routing equation is similar in form to the Muskingum routing equation. Storage is related to both inflow and outflow. However, unlike the Muskingum method, the storage-outflow function is not constrained to being linear. Also, whereas the Muskingum K is a constant, the coefficients reflecting travel time vary with discharge in the variable coefficient routing procedure. The iterative solution required for the variable coefficient method requires more computational effort than the Muskingum or modified Puls methods. However, variable storage coefficient routing should more closely represent an actual flood wave than these methods. Input data for the variable storage coefficient routing procedure consist of cross sections and roughness coefficients.

286. Average lag routing methods. Flood routing by time displacement of average inflow is based strictly on intuition or empirical observations rather than mathematical equations of storage or motion. In the successive average lag method, an outflow hydrograph is computed as the average of the current and previous inflow ordinates. The averaging process is repeated a number of times to produce the outflow hydrograph at the location of interest. The number of times to repeat the averaging is a routing parameter to be determined by calibration. In the progressive average lag method, a number of inflow values are averaged, and the mean value is than lagged by the time of travel of the flood wave to yield the discharge and time of one ordinate of the outflow hydrograph. The process is repeated to determine other ordinates of the outflow hydrograph (USACE 1960).

287. Water-surface profile computations. Flow depths as well as discharges are computed using hydraulic routing techniques. However, hydrologic routing methods typically result in discharge hydrographs. Flow depths are

then computed for a given discharge using either a uniform flow formula, such as the Manning equation, or steady gradually varied water-surface profile procedure. Thus, additional approximations enter the model in converting discharges to flow depths.

288. Computation of water-surface profiles is based on an iterative solution of the one-dimensional energy equation. Head loss is typically estimated using the Manning equation and expansion and contraction coefficients. Weir and orifice equations are used to determine flow and head losses through bridges and culverts. The iterative standard step method is usually used to solve the energy equation. For subcritical flow, the computations begin at a known or assumed surface elevation and proceed upstream. Water-surface profile computations for supercritical flow proceed downstream from a known or assumed upstream water-surface elevation.

Generalized computer programs

289. A number of generalized computer programs have been developed specifically to perform flood wave analysis computations. Flood-routing methods are also included as components of reservoir system simulation models and watershed models. The flood wave analysis capabilities of several leading state-of-the-art computer models are discussed below. These models have been extensively used in modeling reservoir operations.

290. HEC-5. The previously discussed HEC-5 Simulation of Flood Control and Conservation Systems model contains options for the following channel routing methods: modified Puls, Muskingum, working R and D, and average lag. Modified Puls is used for reservoir routing. In the simulation of flood control operations, reservoir releases are determined based on allowable discharges specified for downstream control points. Trial reservoir releases are routed to the downstream control points in an iterative search for the reservoir release which results in the correct discharges at the control points. HEC-5 has an optional capability for computing incremental lateral inflow hydrographs at control points given the total hydrographs.

291. HEC-1. The HEC-1 Flood Hydrograph Package simulates the hydrologic response of a watershed to a flood event (Feldman 1981, HEC 1985). Model capabilities include rainfall-runoff modeling, flood routing, and flood damage evaluation. Like HEC-5, HEC-1 has channel routing options for modified Puls, Muskingum, working R and D, and average lag. HEC-1 also contains a kinematic wave option for watershed and channel routing. For reservoir routing modified Puls is used. Unlike HEC-5, HEC-1 can only handle uncontrolled

reservoirs. Releases can not be made for downstream control points. Flow depth for a given discharge can be computed using the Manning equation assuming uniform flow. HEC-1 has a parameter calibration option which can be used to determine values for the routing parameters for a stream reach, given inflow and outflow hydrographs.

292. HEC-2. The HEC-2 Water-Surface Profile Program computes steady-state gradually varied flow water surface profiles for given discharges (Feldman 1981, HEC 1982b). HEC-2 employs the standard step method to solve the energy equation. HEC-1 and HEC-2 are often used in combination. Discharges are computed with HEC-1, and the corresponding water-surface elevations are computed with HEC-2.

293. MILHY. The Military Hydrology Model (MILHY) performs the same types of computations as HEC-1 and HEC-2. MILHY uses the variable storage coefficient method for channel routing and modified Puls reservoir routing. After computing discharges, water-surface profiles are developed by a standard step method solution of the energy equation. MILHY and HEC-1 are further addressed in the later discussion of watershed modeling.

294. SSARR. The previously discussed Streamflow Synthesis and Reservoir Regulation (SSARR) Model incorporates a hydrologic storage routing method, based on combining a linear relationship between storage and outflow with the continuity equation, which is a variation of Muskingum routing. The SSARR model uses the same approach for both reservoir and channel routing.

295. DWOPER. The Operational Dynamic Wave Model (DWOPER) was developed by the National Weather Service (NWS) for use in the NWS river forecast centers. The generalized unsteady flow computer program is described by Fread (1978). Computations are performed by a weighted four-point implicit finite difference solution of the St. Venant equations. DWOPER is a flexible flood wave analysis model with applicability to river systems of varying physical features, such as irregular geometry, variable roughness parameters, lateral inflows, flow diversions, off-channel storage, local head losses such as bridge contraction-expansions, lock and dam operations, and wind effects. The model can handle any tributary configuration. An automatic calibration feature allows determination of the optimum roughness coefficients from observed hydrographs.

296. DAMBRK. The NWS Dam-Break Flood Forecasting Model (DAMBRK) is described by Fread (1984). DAMBRK is a specific purpose dam-break model that stemmed from the general purpose DWOPER. DAMBRK simulates the failure of the

dam, computes the resultant outflow hydrograph, and simulates the movement of the flood wave through the downstream river valley. Reservoir releases through either uncontrolled or gated spillways can also be simulated. Gate openings can be varied as a function of time.

297. An inflow hydrograph is routed through a reservoir using either hydrologic storage routing or dynamic routing. The type of reservoir routing is a user option. Releases from spillways and outlet works are computed using weir and orifice equations. Two types of breaching may be simulated. An overtopping failure is simulated as a rectangular-, triangular-, or trapezoidal-shaped opening that grows progressively downward from the dam crest with time. Flow through the breach at any instant is calculated using a broad-crested weir equation. A piping failure is simulated as a rectangular orifice that grows with time and can be centered at any specified elevation through the dam. Instantaneous flow through the breach is calculated with either orifice or weir equations, depending on the relation between the elevation of the water surface and the top of the orifice. Weir and orifice flows include corrections that account for tailwater submergence. The pool elevation at which breaching begins, the time required for breach formation, and the beginning and ending geometric parameters of the breach must be specified by the user.

298. The outflow hydrograph from the reservoir is routed downstream using the same finite difference solution of the St. Venant equations contained in DWOPER. Dynamic routing is the only method provided for propagating the wave through the downstream valley. The same dynamic routing algorithm is one of two options provided for reservoir routing. The input data for valley cross sections can specify inactive as well as active flow areas. The inactive portion of a cross section is intended to account for an area where water ponds and/or does not have a significant velocity component in the direction of flow. Inactive areas are considered in the continuity equation but not in the momentum equation.

299. The DAMBRK program can simulate the progression of a dam-break wave through a downstream valley containing one or more additional dams that may or may not fail. However, the multiple dams have to be in series. The model cannot simulate failure of dams on different tributaries. Downstream bridges are simulated in essentially the same way as a dam.

## Watershed Modeling

300. Streamflow hydrographs are basic input data required for the previously discussed simulation modeling categories of reservoir operations, optimization, and flood wave analysis. Historical gaged streamflow data are used to the extent possible. However, the availability of streamflow data is contingent upon gages having been maintained at pertinent locations over a significant period of time. Also, historical streamflow data may not be representative of present and future conditions if significant changes in runoff characteristics have occurred because of urbanization or other land use modifications, construction of reservoirs, variations in water supply withdrawals, or other changed conditions during the period of record. Similarly, if physical characteristics of a basin will change substantially in the future, the historical streamflow record may not provide reliable estimates of future flow conditions. Extreme events, exceeding the most severe event of record, are often of interest in reservoir studies. Precipitation data are more abundant than streamflow data and are not sensitive to changing watershed conditions. Consequently, precipitation data used in conjunction with rainfall-runoff or watershed modeling are commonly used to develop required streamflow data.

### Model components

301. In watershed modeling, a watershed is treated as a system with precipitation being the input to the system and a runoff hydrograph the output. The runoff hydrograph depends upon both precipitation and watershed characteristics. Precipitation-runoff modeling is most often used for developing single-event flood hydrographs but can also be used to develop long-term continuous streamflow sequences. Precipitation-runoff modeling can be divided into three general tasks: (a) compilation of precipitation data, (b) estimation of rainfall excess or direct runoff volume, and (c) development of the runoff hydrograph.

302. Precipitation may be in the form of historical gaged data, real-time measurements, or synthetic storms. Continuous rainfall-runoff modeling may involve many years of gaged hourly rainfall data. Alternatively, gaged rainfall data may be used to reproduce a single historical flood event. Snowfall and snowmelt are also included in many watershed models. Real-time precipitation measurements are used to predict streamflow hydrographs during real-time reservoir operations. Automated hydrometeorological data collection and management systems represent a major area of recent technology advancement

in reservoir operations. Such systems include automated communication of rainfall gage measurements via satellite to a computer system, where a watershed model then predicts streamflows at pertinent locations (Medlock 1985). The Military Hydrology Program is investigating the use of radar measurements of precipitation combined with a rainfall-runoff model for streamflow forecasting. Synthetic storms play a key role in planning and design of reservoir projects. Synthetic storms include the standard project, probable maximum, or statistical storms such as the 50- or 100-year recurrence interval floods.

303. Direct runoff volume or rainfall excess is precipitation volume minus abstractions or losses. Abstractions include interception, depression storage, and infiltration. Viessman et al. (1977) and Linsley, Kohler, and Paulus (1982) provide overview summaries of methods for estimating precipitation abstractions. All of the numerous techniques, which have been developed to compute abstractions, involve significant simplifying assumptions and approximations.

304. The Soil Conservation Service (SCS) curve number method is widely used to compute rainfall excess for ungaged watersheds. This method has an advantage over most other methods in that the parameters can be estimated from watershed characteristics. Other methods may be preferable if measured rainfall and runoff data are available for calibration. The SCS rainfall-runoff relationship (SCS 1972) is as follows:

$$Q = (P - I_a)^2/(P + I_a + S)$$

where runoff (Q), precipitation (P), initial abstraction ($I_a$), and maximum potential retention (S) are all volumes with units of inches. Based on an analysis of empirical data, the SCS estimated $I_a$ to typically be about 0.2S, resulting in the following equation:

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

For computational convenience, the maximum potential abstraction (S) is expressed in terms of a curve number (CN).

$$S = (1000/CN) - 10$$

The SCS (1972, 1975) provides tables developed from empirical analyses which show the relation of CN to soil type, land use, and antecedent moisture conditions.

305. The unit hydrograph concept is commonly used to develop runoff hydrographs for the relatively large watersheds typically involved in reservoir studies. The unit hydrograph has a shape representative of a given basin and rainfall duration and a direct runoff volume of unity. The approach is based on the fundamental assumption of linearity which allows the hydrograph for a given rainfall excess to be computed by multiplying unit hydrograph ordinates by the direct runoff volume. Unit hydrographs can be developed for historical gaged hydrographs. Various alternative methods are also available for developing synthetic unit hydrographs from watershed characteristics in the absence of actual measured streamflow data. Synthetic unit hydrograph methods have been developed by the Soil Conservation Service (1972) and USACE (1959b) as well as others. Viessman et al. (1977) and Linsley, Kohler, and Paulus (1982) outline the computations involved in developing and applying unit hydrographs.

Generalized computer programs

306. A number of the many generalized computer programs available for simulating the rainfall-runoff process are described by the WES (1974) and Fleming (1975). Viessman et al. (1977) describe several watershed models under the categories of rainfall-runoff event simulation (which includes the HEC-1, TR-20, USGS, HYMO, and SWMM models), continuous streamflow simulation (which includes the API, USDAHL, SWM-IV, KWM, OPSET, HSP, TWM, NWSRFS, and SSARR models), and urban runoff simulation (which includes the UCUR, NERO, STORM, RRL, MITCAT, and SWMM models).

307. The MILHY, HEC-1, and SSARR computer programs are briefly described below. All three models were developed within the USACE. HEC-1 and SSARR have been extensively applied in planning, design, and operation of reservoirs both within and outside the USACE. MILHY was developed specifically for military applications of streamflow forecasting.

308. MILHY. The Military Hydrology (MILHY) model was developed as a part of the Military Hydrology Research Program at WES. The microcomputer program is documented by a user's manual (WES 1985b). MILHY was developed for

use by Army Terrain Teams in forecasting streamflows that would result from a given rainfall event over a watershed. Rainfall can be input for a synthetic storm or as measured data from precipitation gages or radar. The following methods are incorporated in the model: SCS curve number procedure for computing losses; two parameter gamma function unit hydrograph; variable storage coefficient channel routing; modified Puls reservoir routing; and standard step water-surface profile computations.

309. HEC-1. The widely used HEC-1 Flood Hydrograph Package was originally released by the Hydrologic Engineering Center (HEC) in 1967. The program has subsequently been expanded and revised several times. HEC-1 is documented by a user's manual (HEC 1985) and described by Feldman (1981). Although limited to mainframe computers in the past, HEC-1 has recently been coded to run on a microcomputer. The HEC-1 package provides a number of options for the various computations involved in simulating the rainfall-runoff process for a flood event and routing flood hydrographs through channels and reservoirs. The package also has automatic parameter calibration, economic flood damage analysis, flood control system optimization, and dam safety analysis capabilities. Rainfall or snowfall can be input as gage data or subbasin averages. The program can also compute three types of synthetic storms: probable maximum precipitation, standard project precipitation, and frequency-based storms. Optional methods for computing runoff volume include the SCS curve number, initial and uniform loss rate, exponential loss rate, and Holtan loss rate methods. The runoff hydrograph can be developed using the unit hydrograph concept or kinematic watershed routing. Synthetic unit hydrograph options include Clark, Snyder, and the SCS dimensionless unit hydrograph.

310. SSARR. The Streamflow Synthesis and Reservoir Regulation (SSARR) model is included in the previous discussion on simulation of reservoir operations. Whereas MILHY and HEC-1 were designed to simulate single or discrete storm events, the SSARR is a "continuous" watershed model which can be used to simulate many years of precipitation and streamflow.

## Synthetic Streamflow Generation

311. Simulation of reservoir systems requires lengthy sequences of streamflow data. Assessing reliability is an aspect of reservoir simulation requiring particularly long sequences of streamflow data. For example,

several hundred equally likely sequences of monthly inflows, with the length of each sequence equal to the expected project life, might be routed through a reservoir to estimate the likelihood of meeting water supply demands. Thus, several thousand years of monthly streamflow data would be needed. Other reservoir modeling applications require more modest amount of streamflow data, but still much more data than is available from historical records. Synthetic streamflow generation techniques have been developed to provide the data needed for reservoir simulation studies.

312. If streamflow can be assumed to be a stationary process (statistical parameters do not change over time) and a reasonably long historical record exists, a statistical model can be used to generate synthetic sequences that reproduce the statistical parameters of the historical data. The basic assumption is that alternative streamflow sequences having the same statistics also have equal probabilities of occurrence. Thus, equally likely streamflow sequences mean that selected statistical parameters are preserved in each sequence.

313. If an adequate historical streamflow record is not available, watershed modeling can be used to convert recorded rainfall data to streamflow. A synthetic streamflow generation model can then be used to extend the computed streamflow data. Synthesis techniques can also be applied to hydrologic times series other than streamflow, such as rainfall, evaporation, and water demands. However, streamflow is the hydrologic variable most often synthesized. Although other time intervals can be used, synthetically generated streamflow data are typically monthly.

314. Books by Shen (1976), Salas et al. (1980), and Bras and Rodriquex-Iturbe (1985) provide a comprehensive treatment of hydrologic time-series analysis and synthesis. The textbooks by Loucks, Stedinger, and Haith (1981) and Linsley, Kohler, and Paulhus (1982) include concise summaries of synthetic streamflow generation. Wurbs et al. (1985) provide an annotated bibliography of references on the topic.

## Models

315. The Markov model is the most widely used approach for synthesizing streamflows. The annual lag-1 Markov model for a single location is expressed as follows:

$$Q_i = \bar{Q} + \rho \ (Q_{i-1} - \bar{Q}) + t_i \ \Delta (1-\rho^2)^{0.5}$$

where $Q_i$ is the synthesized streamflow for year i, $\bar{Q}$ is the mean annual streamflow, $\rho$ is the lag-1 serial correlation coefficient, $t_i$ is a random variable from an appropriate probability distribution with a mean of zero and unit variance, and $\Delta$ is the standard deviation of annual streamflow. This equation states that the flow in period i is the average value from a linear regression of $Q_i$ on $Q_{i-1}$ plus a random element to preserve the variance $\Delta^2$. The parameters $\bar{Q}$, $\Delta$, and $\rho$ are computed from historical data. The random variable $t_i$ is obtained from Monte Carlo sampling. A time series of $Q_i$ is generated by repeatedly applying the Markov model.

316. The monthly (or seasonal) lag-1 Markov model is based on month-to-month autoregression, as follows:

$$Q_{ij} = \bar{Q}_j + \rho_j (\Delta_j/\Delta_{j-1}) (Q_{i-1,j-1} - \bar{Q}_j) + t_i \Delta_j (1-\rho_j^2)^{0.5}$$

where the subscripts i and j refer to year and month (or season), respectively. For monthly data, j varies from 1 through 12 for each year i. The parameters $Q_j$, $\Delta_j$, and $\rho_j$ are computed from historical data for each month (or season). Twelve sets of parameters are required for a monthly model. This model preserves the monthly (seasonal) values of the statistical parameters in the generated streamflow sequences.

317. Another approach to monthly or seasonal streamflow synthesis is to disaggregate generated annual flows. The disaggregation approach is particularly advantageous for generating streamflow data for multiple locations. Lane (1985) has developed a practical disaggregation model which preserves the most important features of the correlations between seasonal flows at multiple sites.

318. The autoregressive Markov model discussed above is the simplest of a flexible family of autoregressive moving average (ARMA) time-series models sometimes called Box-Jenkins models. ARMA models have been widely used in the modeling and forecasting of time series in many fields besides water resources. ARMA and other more mathematically sophisticated time-series models are addressed in the previously cited references.

### Generalized computer programs

319. The HEC-4 and LAST computer packages developed by the Hydrologic Engineering Center and Bureau of Reclamation, respectively, generate synthetic streamflows utilizing the concepts discussed above. Input consists of historical streamflow data. Output is a sequence of synthetically generated streamflows which preserve the statistical parameters of the input data.

320. HEC-4. The HEC-4 Monthly Streamflow Simulation computer program generates monthly streamflow data using the lag-1 Markov model (HEC 1971, Feldman 1981). Streamflows are assumed to fit a log-Pearson Type III probability distribution. Computations can be for a single location or several interrelated locations. For multiple locations, the generated flow is computed from a regression relationship of the current month at all other stations, the previous month deviates at all other stations, and a random component proportional to the unexplained variance. The program will also reconstruct missing streamflows on the basis of concurrent streamflow at other stations and the statistical relationship between the stations.

321. LAST. The Applied Stochastic Techniques (LAST) package generates annual, seasonal, and/or monthly streamflow data at a number of interrelated stations (Lane 1985, Frevert and Lane 1985). The model preserves year-to-year serial correlations with the multilag autoregressive Markov model in addition to seasonal serial correlations. Cross correlations between locations are also preserved. Flows are generated at key stations using a lag-1 or lag-2 Markov model. The annual flows at the selected key stations provide the basis for estimating flows at other locations of interest. The annual flows are then disaggregated to seasonal flows.

## Water Quality Modeling

322. The physical, chemical, and biological characteristics of water in a reservoir-stream system change with time and location. Silt and debris are carried by surface waters, often resulting in muddy or turbid streams. Minerals picked up by surface runoff become a component of streamflow. Plants and algae grow in reservoirs and other areas of stagnant water. Surface waters are used for the disposal of most of the world's liquid wastes. Wastes have a major impact on water quality and add greatly to the spectrum of impurities present. Evaporation concentrates the impurities. Mathematical models

provide a means to predict the impacts of natural processes and activities of man on the water quality of a reservoir-stream system.

Models

323. A water quality model is a mathematical statement or set of statements that equate water quality at a point of interest to causative factors (Viessman et al. 1977). In general, water quality models are designed to: (a) accept as input constituent concentration versus time at points of entry to the system, (b) simulate the mixing and reaction kinetics of the system, and (c) synthesize a time-distributed output at the system outlet. Either stochastic or deterministic approaches may be taken in developing methods for predicting pollution loads. A stochastic model is based on determining the likelihood of a particular output quality response by statistical means. A deterministic model relates water quality to a known or assumed hydrologic input.

324. Water quality constituents can be categorized as organic, inorganic, radiological, thermal and biological, or they can be subdivided into specific forms such as biochemical oxygen demand, nitrogen, phosphorus, and so forth. Pollutants typically of interest include silt, pesticides, fertilizers, fecal organisms, nitrates, and phosphates. Unstable pollutants, such as biochemical oxygen demand, radioactive wastes, and heat that have a time-dependent decay, are classified as nonconservative. Many inorganic pollutants are treated as being conservative.

325. Modeling water quality in streams and reservoirs is a complex topic addressed extensively in the literature. Orlob (1983) and Tchobanoglous and Schroeder (1985) provide a good starting point for studying water quality and water quality modeling.

Generalized computer programs

326. During the 1960's and early 1970's, mathematical modeling of reservoir-stream systems focused on temperature, dissolved oxygen, and biochemical oxygen demand. The USACE developed several generalized models for reservoir temperature analysis during this period that continue to be widely used within and outside the USACE (HEC 1972, North Pacific Division 1973, Baltimore District 1977, WES 1980). By the mid-1970's the need was apparent for developing capabilities for analyzing a more comprehensive range of water quality parameters. Two major water quality computer packages developed by the Hydrologic Engineering Center are described below.

327. WQRRS. The Water Quality for River - Reservoir Systems (WQRRS) package was developed for the Hydrologic Engineering Center by private consulting firms (HEC 1978, Feldman 1981). The purpose of the model is to simulate the impact of water management projects on the water quality of rivers and reservoirs. The model is composed of three separable modules: stream hydraulics, river quality, and reservoir quality. Two postprocessor programs are also available which compute statistical summaries and plot graphs of the results.

328. A reservoir is represented in the reservoir transport module by a series of one-dimensional horizontal slices. The volume of water within a horizontal slice is assumed to be fully mixed. External inflows or withdrawals occurring within a slice are instantaneously mixed throughout the slice from the headwaters to the impounding structure. The transfer of heat and mass between slices can occur by advection and diffusion. The diffusion mechanism represents molecular and turbulent diffusion as well as convective mixing. The movement of water in the lake is determined by the location and rates on inflow and outflow. Inflow can be entrained in all slices in proportion to their size or left to seek the slice of like density. Withdrawal from the reservoir can be made by two methods, one which automatically computes the zone (slices) of influence and the other which uses the Corps selective withdrawal.

329. The river transport module, also known as the Stream Hydraulics Package (HEC 1979), performs a one-dimensional routing of the flow from one subreach to the next. The river reaches used are the traditional representation of fully mixed lengths of channel. Thus, the river module divides the water body into a series of vertical segments and the reservoir module into horizontal segments. The river module may be appropriate to simulate nonstratified impoundments with rapid flow through. The river module has six methods for routing the flows: steady-state backwater, St. Venant unsteady flow, kinematic wave, Muskingum, modified Puls, or direct input stage versus flow relationship.

330. The water quality simulation portrays the important processes which determine the thermal and quality characteristics of water bodies. Each chemical and biological component is expressed as a function of conservation of mass and kinetic principles. All chemical and biological processes are assumed to occur in an aerobic environment. A partial differential equation representing the dynamics of heat and biotic and abiotic material is used for

temperature and other constituents passively transported with the movement of water. A modification is made for those components which are fixed to the bottom or are mobile (i.e. fish). Water temperatures are governed by external heat fluxes. Sources and sinks of water quality constituents include settling, first-order decay, aeration, chemical transformation, biologic uptake and releases, growth respiration, and mortality including predation.

331. The main biologic and chemical constituents considered in the water quality simulation are fish, aquatic insects, benthic animals, zooplankton, phytoplankton, benthic algae, detritus, organic sediment, inorganic suspended solids, inorganic sediment, inorganic carbon, dissolved phosphate, ammonia, nitrites, nitrates, oxygen, coliform bacteria, total alkalinity, total dissolved solids, pH, and unit toxicity. The ecologic processes in a lake environment are centered around phytoplankton (algae). The ecologic processes in the stream model are centered around benthic algae as the critical link in the flood chain.

332. The input data requirements are quite extensive, as evidenced by the complexity of the hydraulic and water quality processes previously described. Hydrologic, meteorologic, hydraulic, ecologic, and water chemistry data are required. The program produces output information for all these items at desired times and locations. The three modules (reservoir, river, and quality) are run in sequence with the output of one being input to the next.

333. HEC-5Q. The HEC-5Q Simulation of Flood Control and Conservation Systems Including Water Quality Analysis computer program is a version of the previously discussed HEC-5 with water quality options added (HEC 1984; Willey, Smith, Duke 1985). Work was initiated in 1979 to modify HEC-5 to evaluate reservoir system operations for water quality control. HEC-5Q consists of the HEC-5 flow simulation module and an added water quality module.

334. HEC-5Q provides the capability to analyze water temperature and up to three conservative and three nonconservative constituents selected by the user. Dissolved oxygen can also be analyzed if at least one of the constituents is an oxygen-demanding parameter.

335. The water quality simulation module accepts system flows generated by the flow simulation module and computes the distribution of all the water quality constituents in up to ten reservoirs and their associated downstream stream reaches. The model also selects gate openings for reservoir selective withdrawal structures to meet user-specified water quality objectives at

downstream control points. If the water quality objectives cannot be satisfied by the flow module computed flows, the model computes the increase in flow necessary to meet the objectives.

336. Reservoirs are represented conceptually by series of one-dimensional horizontal slices. Within each slice or volume element, the water is assumed to be fully mixed. The stream system is represented conceptually as a linear network of segments or volume elements.

## Sediment Transport

337. A natural river continually changes with reference to its position on the floodplain, meander pattern, and cross-section shape and size. Nature maintains a delicate balance between the water flowing in a river, the sediment load moving with the water, and the material forming the streambed. A qualitative expression of this balance is provided by Lane's relation:

$$Q_s \times Q_{50} \sim Q \times S$$

where bed material load ($Q_s$) times sediment size ($Q_{50}$) is proportional to water discharge (Q) times the energy gradient (S). A channel is maintained in dynamic equilibrium by balancing changes in the sediment load and sediment size, with compensating changes in the water discharge and the energy gradient. Various empirical equations have been developed based on these variables.

338. Nature's balance is upset whenever man's activity changes any of the following factors: water yield from the watershed, sediment yield from the watershed, water discharge duration curve, size of sediment particles, or the depth, velocity, slope, or width of flow. Constructed works which impede the natural meandering of a stream also upset the balance. The objective of most sediment studies is to evaluate the impact on the flow system from changing any of these factors.

339. Construction of a reservoir changes the hydraulics of flow by drastically decreasing the energy gradient, with a resulting loss of sediment transport capability. A delta is formed in the upper reaches of a reservoir as disposition occurs. The smaller the particles, the farther they will move

into the reservoir before depositing. Sediment deposits result in a depletion of reservoir storage capacity.

340. Trapping of sediment load by a reservoir changes the downstream sediment transport conditions. The reduction in sediment load, especially bed material load, causes the energy in the flow to be out of balance with the boundary material for the downstream channel. Degradation of the channel results. Initially, the degradation will be concentrated a short distance below the dam as the equilibrium sediment load is reestablished. The degradation trend will migrate downstream with time. The extent of degradation is complicated by the fact that the reservoir also changes the water discharge duration relation. Control of floods may actually have the opposite effect of causing aggradation.

341. The analysis of erosion and sediment transport is much more complex than fixed bed hydraulics. Theory regarding interactions between flowing water and a movable boundary is limited and incomplete. Analysis methods are very approximate. The state of the art of sediment transport technology is outlined by Simons and Senturk (1977) and Thomas (1977).

342. The HEC-6 Scour and Deposition in Rivers and Reservoirs computer package is documented by a user's manual (HEC 1977b) and described by Feldman (1981). HEC-6 simulates the interaction between the sediment material forming the river bed, the water-sediment mixture, and the hydraulics of flow. The primary purpose of the model is to simulate the dynamic scour and deposition process along rivers. Sediment deposition in deep reservoirs can also be computed.

343. Reservoir deposition can be analyzed to determine both the volume and location of sediment deposits. Degradation of the streambed downstream from a dam can also be simulated. Long-term trends of scour and deposition in a stream channel resulting from modifications in flow frequency and duration can be studied using the model. Channel contraction required to either maintain navigation depths or reduce the volume of maintenance dredging can be analyzed, but not in the detail provided by moveable bed physical model studies. The influence of dredging on the rate of deposition can be studied. Scour during floods can be predicted.

344. Input data include: reservoir and channel cross sections, discharge and stage hydrographs, inflowing sediment load and gradation, gradation of the bed material, and properties of the fluids and sediment. The program output describes the change in the bed elevation and the sediment transport

for desired points in time. The one-dimensional model considers only scour and deposition in the main channel. Meanders or lateral changes in bed shape can not be analyzed.

## PART VII: SUMMARY AND CONCLUSIONS

345. Rivers throughout the world are subject to a large degree of regulation by dams and reservoirs. Streamflow conditions depend upon man's operation of control structures as well as nature's provision of precipitation. With rapidly increasing world population, effective surface water management is crucial to providing needed supplies of water, food, and electrical energy as well as transportation and other services. Dams are essential facilities for controlling and utilizing a surface water resource. Dams are also potential targets for acts of war or terrorism. Induced flooding below many large dams in the world could be catastrophic. Reservoir releases can provide effective barriers during combat operations. Most of the over 35,000 dams with heights exceeding 10 m were constructed since World War II. The size as well as the number of dams has drastically increased in recent decades. Consequently, dams have assumed an increasingly important role in military hydrology.

346. This report provides a general overview of dams, their operation, and associated modeling capabilities. Mathematical models have been extensively used by the civilian sector to study a broad range of reservoir operation problems. The same models could be adapted to military needs. Potential military as well as civilian applications are broad in scope involving a variety of types of models. The present report discusses models in the categories of simulation, optimization, flood wave analysis, streamflow synthesis, water quality, and sediment transport. Development of rating curves is important to each of these types of models as well as real-time operations and is covered as a separate topic.

## PART VIII: RECOMMENDATIONS

347. This report represents a portion of a larger effort, i.e., the prediction of obstacles created by reservoirs. Much additional work is needed. Some fruitful areas of research are as follows:

- a. Determination of the estimated time of effectiveness of obstacles created by reservoir regulation is essential for proper evaluation of a river as an obstacle. The obstacle effectiveness is not only a function of the hydraulics as the flood wave passes through the downstream valley, but also a function of the impact on mobility through the floodplain. The soil moisture that is retained after flooding will dictate when vehicles can once again cross the floodplain. Despite the crucial importance in obstacle simulation, this aspect has not been addressed adequately.
- b. Prediction of damage to bridges and other structures located in the floodplain is required to accurately predict the extent to which reservoirs can create and maintain an obstacle.

## References


Baltimore District, US Army Corps of Engineers. 1977. "Thermal Simulation of Lakes, User's Manual," Baltimore, Md.

Biswas, A. K., and Chatterjee, S. 1971 (Mar). "Dam Disasters - An Assessment," Engineering Journal.

Bras, R. L., and Rodriquez-Iturbe, I. 1985. Random Functions and Hydrology, Addison-Wesley Publishing Company.

Brater, E. F., and King, H. W. 1976. Handbook of Hydraulics, 6th ed., McGraw-Hill, New York.

Browder, Lewis E. 1978. "RESOP-II Reservoir Operating and Quality Routing Program, Program Documentation and User's Manual," UM 20, Texas Department of Water Resources, Austin, Tex.

Chow, Ven Te. 1959. Open-Channel Hydraulics, McGraw-Hill, New York.

Coomes, R. T. 1981. "Regulation of Arkansas Basin Reservoirs," Proceedings of the National Workshop on Reservoir Systems Operations, American Society of Civil Engineers, New York.

Copley, Ross. 1981. "Arkansas River Systems Regulation Study," Proceedings of the National Workshop on Reservoir Systems Operations, American Society of Civil Engineers, New York.

Cunge, J. A. 1969. "On the Subject of Flood Propagation Computation Method (Muskingum Method)," Journal of Hydraulic Research, Vol 7, No. 2, pp 205-230.

Cunge, J. A., Holley, F. M., and Verway, A. 1980. Practical Aspects of Computational River Hydraulics, Pitman Publishers, London, England.

Davis, C. V., and Sorensen, K. E., eds. 1984. Handbook of Applied Hydraulics, 3rd ed., McGraw Hill, New York.

Dziuban, S. W. 1945. "Rhine River Flood Prediction Service," The Military Engineer, American Society of Military Engineers, Vol XXXVII, No. 239, pp 348-353.

__________. 1946. "Hydraulics Model Experiments, Planning for Crossing the Rhine," The Military Engineer, American Society of Military Engineers, Vol XXXVIII, No. 247, pp 189-193.

__________. 1947. "Employment of Artificial Floods in Military Operations," The Military Engineer, American Society of Military Engineers, Vol XXXIX, No. 261, pp 279-284.

__________. 1949a. "Strategic Flooding in the United States," The Military Engineer, American Society of Military Engineers, Vol XLI, No. 284, pp 439-443.

__________. 1949b. "Techniques of Artificial Flooding in Military Operations," The Military Engineer, American Society of Military Engineers, Vol XLI, No. 283, pp 342-344.

Dziuban, S. W. 1950. "Implications of Artificial Flooding in Military Operations," The Military Engineer, American Society of Military Engineers, Vol XLII, No. 285, pp 13-15.

Eichert, B. S. 1979. "Reservoir Storage Determination by Computer Simulation of Flood Control and Conservation Systems," USACE Hydrologic Engineering Center, Technical Paper No. 66, Davis, Calif.

Eicher, B. S., Peters, J. C., and Pabst, A. F. 1975. "Techniques for Real-time Operations of Flood Control Reservoirs in the Merrimack River Basin," Technical Paper No. 45, USACE Hydrologic Engineering Center, Davis, Calif.

Feldman, A. D. 1981. "HEC Models for Water Resources Systems Simulation," Advances in Hydroscience, Vol 12, edited by Ven Te Chow, Academic Press, New York.

Fleming, G. 1975. Computer Simulation Techniques in Hydrology, American Elsevier Publishing Company, New York.

Ford, D. T., Garland, R., and Sullivan, C. 1981. "Operation Policy Analysis: Sam Rayburn Reservoir," Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Vol 107, No. WR2, pp 339-350.

Fread, Danny L. 1978. "National Weather Service Operational Dynamic Wave Model," Office of Hydrology, National Weather Service, Silver Springs, Md.

__________. 1982. "Flood Routing: A Synopsis of Past, Present, and Future Capability," Rainfall-Runoff Relationship, V.P. Singh, ed., Water Resources Publications, Littleton, Colo.

__________. 1984. "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Office of Hydrology, National Weather Service, Silver Spring, Md.

Frevert, D. K., and Lane, W. L. 1985. "Development and Application of a Two-Level Spatial Disaggregation Procedure for the Last Statistical Hydrology Package," Fourth International Hydrology Symposium, Fort Collins, Colo.

Gilbert, K. D., and Shane, R. M. 1982. "TVA Hydro Scheduling Model: Theoretical Aspects," Journal of Water Resources Planning and Management Division, American Society of Civil Engineers, Vol 108, No. 1, pp 21-36.

Golze, A. R., ed. 1977. Handbook of Dam Engineering, Van Nostrand Rheinhold Co., New York.

Hufschmidt, M. F. and Fiering, M. B. 1966. Simulation Techniques for Design of Water Resource Systems, Harvard University Press, Cambridge, Mass.

Hula, R. L. 1981. "Southwestern Division Reservoir Regulation Simulation Model," Proceedings of the National Workshop on Reservoir Systems Operations, edited by Toebes and Sheppard, American Society of Civil Engineers, New York.

Hydrologic Engineering Center, US Army Corps of Engineers. 1971. "HEC-4 Monthly Streamflow Simulation, User's Manual," Davis, Calif.

__________. 1972. "Reservoir Temperature Stratification, User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1976. "Hydrologic Engineering Methods for Water Resources Development, Vol 7, Reservoir Operation for Flood Control," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1977a. "Hydrologic Engineering Methods for Water Resources Development, Vol 9, Reservoir System Analysis for Conservation," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1977b. "HEC-6 Scour and Deposition in Rivers and Reservoirs, User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1978. "Water Quality for River-Reservoir Systems, Computer Program Description," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1979. "Stream Hydraulics Package, User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1981. "HEC-3 Reservoir System Analysis for Conservation, User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1982a. "HEC-5 Simulation of Flood and Conservation System, User's Manual, Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1982b. "HEC-2 Water-Surface Profiles, User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1984. "HEC-5Q Simulation of Flood Control and Conservation Systems Including Water Quality Analysis, Draft User's Manual," Davis, Calif.

Hydrologic Engineering Center, US Army Corps of Engineers. 1985. "HEC-1 Flood Hydrograph Package, User's Manual," Davis, Calif.

International Commission on Large Dams, 1984. "World Register of Dams," Paris, France.

Jansen, R. B. 1980. Dams and Public Safety, US Government Printing Office, Denver, Colo.

Johnson, F. A., and Illes, P. 1976 (Dec). "A Classification of Dam Failures," Water Power and Dam Construction, pp 43-45, H.E. Warne Ltd., London, England.

Lane, W. L. 1985. "Applied Stochastic Techniques (LAST), User Manual," Bureau of Reclamation, Engineering and Research Center, Denver, Colo.

Linsley, R. K., and Franzini, J. B. 1979. Water Resources Engineering, 3rd ed., McGraw-Hill, New York.

Linsley, R. K., Kohler, M. A., and Paulhus, J. L. H. 1982. Hydrology for Engineers, 3rd ed., McGraw Hill, New York.

Loucks, D. P., Stedinger, J. R., and Haith, D. A. 1981. Water Resource Systems Planning and Analysis, Prentice-Hall, Englewood Cliffs, New Jersey.

Maass, A., Hufschmidt, M. M., Dorfman, R., Thomas, H. A., Marglin, S. A., and Fair, G. M. 1966. Design of Water Resource Systems, Harvard University Press, Cambridge, Mass.

Manzer, D. F. and Barnett, M. P. 1966. "Analysis by High-Speed Digital Computer," Design of Water Resource Systems, Harvard University Press, Cambridge, Mass.

Martin, Quentin W., 1981. "Surface Water Resources Allocation Model (AL-V), Program Documentation and User's Manual," UM-35, Texas Department of Water Resources, Austin, Tex.

__________. 1982. "Multivariate Simulation and Optimization Model (SIM-V), Program Documentation and User's Manual," UM-38, Texas Department of Water Resources, Austin, Tex.

__________. 1983. "Optimal Operation of Multiple Reservoir Systems," Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Vol 109, No. 1.

McMahon, G. F., Bonner, V., and Eichert, B. S. 1979. "Operational Simulation of a Reservoir System with Pumped Storage," USACE Hydrologic Engineering Center, Technical Paper No. 60, Davis, Calif.

McMahon, T. A. and Main R. G. 1978. "Reservoir Capacity and Yield," Developments in Water Science, Vol 9, Elsevier Scientific Publishing, New York.

Medlock, J. M. 1985. "Geostationary Operational Environmental Satellites (GOES) Data Collection Procedure - Fort Worth District," Hydraulics and Hydrology in the Small Computer Age (edited by W.R. Waldrop), American Society of Civil Engineers, New York.

Mermel, T. W. 1979 (Nov). "Major Dams of the World," Water Power and Dam Construction, pp 95-105, London, England.

North Pacific Division, US Army Corps of Engineers. 1973. "WRE Reservoir Temperature Simulation Model, User's Manual," Portland, Ore.

__________. 1975. "Program Description and User Manual for Streamflow Synthesis and Reservoir Regulation Model," Portland, Ore.

Orlob, G. T., ed. 1983. Mathematical Modeling of Water Quality, Wiley Interscience.

Palmer, R. N., Wright, J. R., Smith, J. A., Cohon, J. L., and ReVelle, C. S. 1980. "Policy Analysis of Reservoir Operation in the Potomac River Basin," 3 vol, John Hopkins University, Department of Geography and Environmental Engineering, Baltimore, Md.

__________. 1982. "Reservoir Management in Potomac River Basin," Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Vol 108, No. WR1.

Sakkas, J. G. 1980. "Dimensionless Graphs of Floods from Ruptured Dam," Research Note No. 8, Hydrologic Engineering Center, US Army Corps of Engineers, Davis, Calif.

Salas, J. D., Delleur, J. S., Yevjevich, V., and Lane, W. L. 1980. Applied Modeling of Hydrologic Time Series, Water Resources Publications, Fort Collins, Colo.

Schnitter, N. J. 1967. "A Short History of Dam Engineering," Water Power, Tothill Press Ltd., London, Vol 19, No. 4, pp 142-148.

Shen, H. W., ed. 1976. Stochastic Approaches to Water Resources, Water Resources Publications, Fort Collins, Colo.

Sigvaldason, O. T. 1976. "A Simulation Model for Operating a Multipurpose Multireservoir System," Water Resources Research, Vol 12, No. 2.

Simons, D. B. and Senturk, F. 1977. Sediment Transport Technology, Water Resources Publications, Fort Collins, Colo.

Smith, N. 1971. A History of Dams, Citadel Press, Secaucus, N.J.

__________. 1976. Man and Water, A History of Hydro-Technology. Peter Davies Limited, London, England.

Soil Conservation Service, US Department of Agriculture. 1972. "National Engineering Handbook, Section 4 - Hydrology," Washington, DC.

__________. 1975. "Urban Hydrology for Small Watersheds," Technical Release 55.

Strezepek, K. M., and Lenton, R. L. 1978. "Analysis of Multipurpose River Basin Systems: Guidelines for Simulation Modeling," Massachusetts Institute of Technology, Report No. 236.

Strezepek, K. M., Rosenberg, M. S., Goodman, D. D., Lenton, R. L., and Marks, D. M. 1979. "User's Manual for MIT River Basin Simulation Model," Massachusetts Institute of Technology, Civil Engineering Department, Report No. 242.

Sullivan, G. J. 1989. "Reservoir Analysis Model for Battlefield Operations," Technical Report EL-89-7, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Tchobanoglous, G., and Schroeder, E. D. 1985. Water Quality, Addison-Wesley.

Texas Water Development Board. 1974. "Analytical Techniques for Planning Complex Water Resource Systems," Report 183, Austin, Tex.

Thomas, H. H. 1976. The Engineering of Large Dams, 2 Vol, John Wiley and Sons, London, Great Britain.

Thomas, W. A. 1977. "Hydrologic Engineering Methods for Water Resources Development, Volume 12, Sediment Transport," Hydrologic Engineering Center, Davis, Calif.

United Nations. 1978. "Register of International Rivers," Pergamon Press, Oxford, England.

US Army Corps of Engineers. 1959a. "Reservoir Operation," Engineering Manual 1120-2-3600, Washington, DC.

US Army Corps of Engineers. 1959b. "Flood Hydrograph Analysis and Applications," Engineering Manual 1110-2-1405, Washington, DC.

__________. 1960. "Routing of Floods Through River Channels," Engineering Manual 1110-2-1408, Washington, DC.

__________. 1965. "Standard Project Flood Determinations," Engineering Manual 1110-2-1411, Washington, DC.

US Army Corps of Engineers, Office of the Chief of Engineers. 1963 (Aug). "Hydraulic Design of Reservoir Outlet Structures," EM 1110-2-1602, Washington, DC.

__________. 1965 (Mar). "Hydraulic Design of Spillways," EM 1110-2-1603, Washington, DC.

US Bureau of Reclamation. 1976. Design of Gravity Dams, Denver, Colo.

__________. 1977a. Design of Arch Dams, Denver, Colo.

__________. 1977b. Design of Small Dams, Denver, Colo.

Viessman, W., Knapp, J. W., Lewis, G. L., and Harbaugh, T. E. 1977. Introduction to Hydrology, 2nd ed. Harper and Row, New York.

Viessman, W., and Welty, C. 1985. Water Management, Technology and Institutions, Harper and Row Publishers, New York.

Wagner, H. M. 1975. Principles of Operations Research, Prentice-Hall, Englewood Cliffs, New Jersey.

Waterways Experiment Station, US Army Corps of Engineers. Undated. "Hydraulic Design Criteria," Vicksburg, Miss.

__________. 1974. "Models and Methods Applicable to Corps of Engineers Urban Studies," Miscellaneous Paper H-74-8, Vicksburg, Miss.

__________. 1980. "WESTEX-A Reservoir Heat Budget Model, Draft Computer Program Description," Vicksburg, Miss.

__________. 1985a. Engineering Computer Programs Library Catalog, Vicksburg, Miss.

__________. 1985b. "MILHY User Manual," Military Hydrology Team, Vicksburg, Miss.

Wetmore, J. N., and Fread, D. L. 1981. "The NWS Simplified Dam-break Flood Forecasting Model," Proceedings of the 5th Canadian Hydrotechnical Conference, Fredericton, New Brunswick, Canada, 26-27 May 1981.

Willey, R. G., Smith, D. J., and Duke, J. H. 1985. "Modeling Water Resources Systems for Water Quality," Technical Paper No. 104, Hydrologic Engineering Center, Davis, Calif.

Williams, J. R. 1969. "Flood Routing with Variable Travel Time or Variable Storage Coefficients," Transactions of the American Society of Agricultural Engineers, Vol 12, No. 1.

Wurbs, R. A. 1985. "Military Hydrology; Report 9: State-of-the-Art Review and Annotated Bibliography of Dam-Breach Flood Forecasting," Miscellaneous Paper EL-79-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

__________. 1986. "Military Hydrology; Report 13: Comparative Evaluation of Dam Breach Flood Forecasting Methods," Miscellaneous Paper EL-79-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Wurbs, R. A., Tibbets, M. N., Cabezas, L. M., and Roy, L. C. 1985. "State-of-the-Art Review and Annotated Bibliography of Systems Analysis Techniques Applied to Reservoir Operation," Texas Water Resources Institute, Technical Report No. 136, College Station, Tex.

Yeh, W. W-G. 1982. "State-of-the-Art Review: Theories and Applications of Systems Analysis Techniques to the Optimal Management and Operation of a Reservoir System," University of California at Los Angeles, School of Engineering and Applied Science.